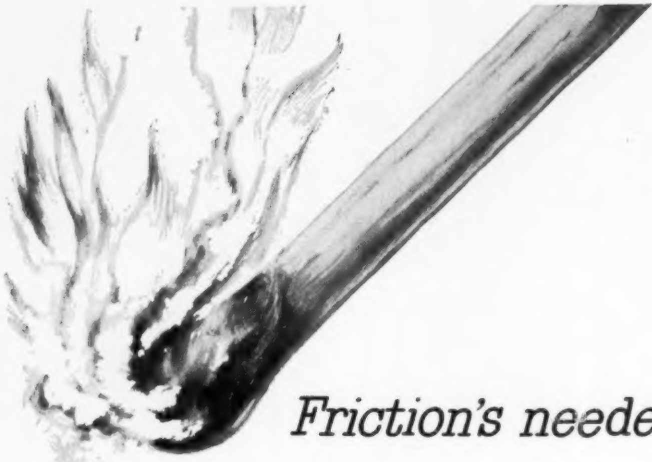
FEBRUARY 1954

Journal

VOL. 46 • NO. 2

AMERICAN WATER WORKS ASSOCIATION

In this issue



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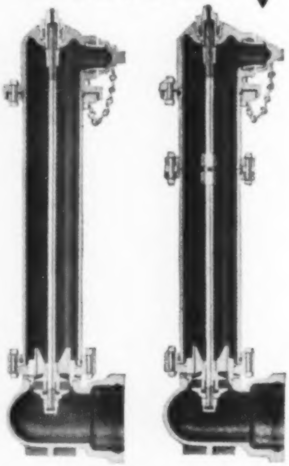

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AMERICAN WATER WORKS ASSOCIATION

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Contents

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Coming Meetings

AWWA SECTIONS

Feb. 10–12—Indiana Section at Lincoln Hotel, Indianapolis. Secretary, G. G. Fassnacht, State Dept. of Health, 1330 W. Michigan St., Indianapolis 7, Ind.

Feb. 16—New Jersey Section Winter Luncheon Meeting, at Essex House, Newark, N.J. Secretary, C. B. Tygert, Box 178, Newark 1, N.J.

Mar. 17–19—Illinois Section at La-Salle Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 122 S. Michigan Ave., Chicago 3, Ill.

Mar. 18—New England Section at Hotel Statler, Boston. Secretary, George G. Bogren, Partner, Weston & Sampson, 14 Beacon St., Boston 8, Mass.

Mar. 29–31—Southeastern Section at Poinsette Hotel, Greenville, S.C. Secretary, N. M. deJarnette, 245 State Office Bldg., Atlanta 3, Ga.

Apr. 7–9—Kansas Section at Broadview Hotel, Emporia. Secretary, Harry W. Badley, Repr., Neptune Meter Co., 119 W. Cloud, Salina, Kan.

Apr. 12–14—Canadian Section at Royal York Hotel, Toronto. Secretary, A. E. Berry, Director, Ontario Dept. of Health, Parliament Bldgs., Toronto 8, Ont.

Apr. 22–23—Nebraska Section at Cornhusker Hotel, Lincoln. Secretary, E. Bruce Meier, Asst. Prof. of Civil Engineering, Univ. of Nebraska, Lincoln, Neb.

Apr. 22–23—New York Section at Woodruff Hotel, Watertown. Secretary, Kimball Blanchard, Rm. 1922, 50 W. 50 St., New York 20, N.Y.

AWWA ANNUAL CONFERENCE

Seattle, Wash. **May 23-28, 1954**

All reservations are being cleared through the AWWA office. The hotels have agreed to accept no reservations for the 1954 Conference except as they are requested on the standard form, through the AWWA.

(Continued on page 8)

Lithographed on stone for U. S. Pipe and Foundry Co. by John A. Noble, A. N. A.

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Coming Meetings

(*Continued from page 6*)

Apr. 22–24—Arizona Section in Tucson. Secretary, M. V. Ellis, Supervisor, Sewage Treatment Plant, Phoenix, Ariz.

Apr. 23–24—Montana Section at Baxter Hotel, Bozeman. Secretary, A. W. Clarkson, Acting Chief, Water Section, Div. of Environmental Sanitation, State Board of Health, Helena, Mont.

May 25—Pacific Northwest Section luncheon meeting at Norselander Cafe, Seattle, Wash. (Business meeting only; to be held during AWWA Conference May 23–28.)

Jun. 16–18—Pennsylvania Section at Americus Hotel, Allentown. Secretary, L. S. Morgan, Div. Engr., State Dept. of Health, Greensburg, Pa.

Jun. 22—New Jersey Section Summer Outing (inspection trip and luncheon) at Boonton. Secretary, C. B. Tygert, Box 178, Newark 1, N.J.

FALL SECTION MEETINGS

Sept. 9–10—New York, in Montauk, L.I.
Sept. 15–17—Michigan, in Muskegon
Sept. 20–22—Kentucky-Tennessee, in Nashville
Sept. 22–24—Ohio, in Dayton
Sept. 26–28—Missouri, in Jefferson City
Sept. 28–30—Wisconsin, in Green Bay
Oct. 13–15—Iowa, in Cedar Rapids
Oct. 17–20—Southwest, in El Paso, Tex.
Oct. 24–27—Alabama-Mississippi, in Birmingham
Oct. 26–29—California, in Long Beach
Oct. 27–29—Chesapeake, in Baltimore
Nov. 3–5—Virginia, in Richmond
Nov. 4–6—New Jersey, in Atlantic City
Nov. 7–10—Florida, in St. Petersburg
Nov. 8–9—West Virginia, in Huntington
Nov. 8–10—North Carolina

OTHER ORGANIZATIONS

Feb. 1–5—Corrosion Short Course, at Washington State College, Pullman, Wash. Sponsored by NACE and Div. of Industrial Services, Washington Inst. of Technology. Details from E. B. Parker, Director, Div. of Industrial Services, State College of Washington, Pullman, Wash.

Feb. 11–12—Ohio Water Clinic Conference, at Ohio State Univ., Columbus.

Mar. 10–12—Short Course for Water and Sewerage Personnel, at Louisiana State Univ., Baton Rouge. Details from John H. O'Neill, Director, Div. of Public Health Eng., Dept. of Health, Civil Courts Bldg., New Orleans 7, La.

Mar. 15–19—National Assn. of Corrosion Engineers, in Kansas City, Mo.

Apr. 21–23—Southern Industrial Wastes Conference, at Hotel Shamrock, Houston, Tex. Sponsored by Southern Assn. of Science & Industry and Texas Chemical Council.

May 4–6—American Public Power Assn., in Chicago

May 4–7—American Welding Society, in Buffalo, N.Y.

Jun. 13–18—American Society for Testing Materials, in Chicago

Jul. 25–31—Inter-American Conference of Sanitary Engineering, in Sao Paulo, Brazil.

Oct. 4–6—New England Water Works Assn., Poland Springs, Me.

Oct. 11–14—Federation of Sewage & Industrial Wastes Assn., in Cincinnati.

Oct. 11–15 (tentative)—American Public Health Assn., in Buffalo

Oct. 17–20—American Society of Civil Engineers, in New York City

Oct. 19–22—Pennsylvania Water Works Assn., in Atlantic City, N.J.

Nov. 17–19—Water Works Management Short Course, at Univ. of Illinois, Allerton Park, Ill.

Nov. 28–Dec. 3—American Society of Mechanical Engineers, in New York


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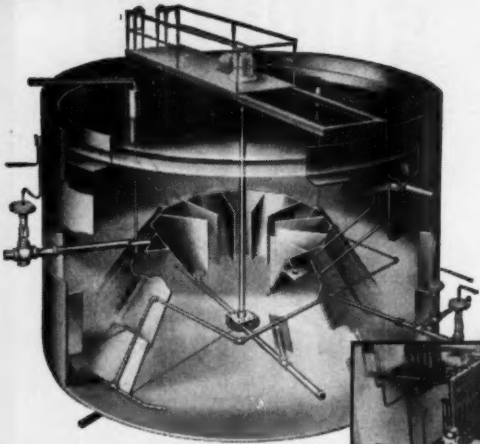

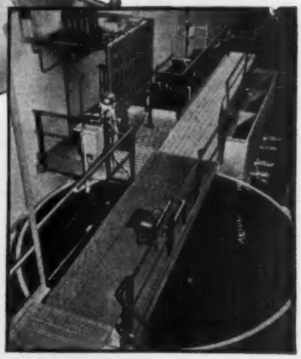

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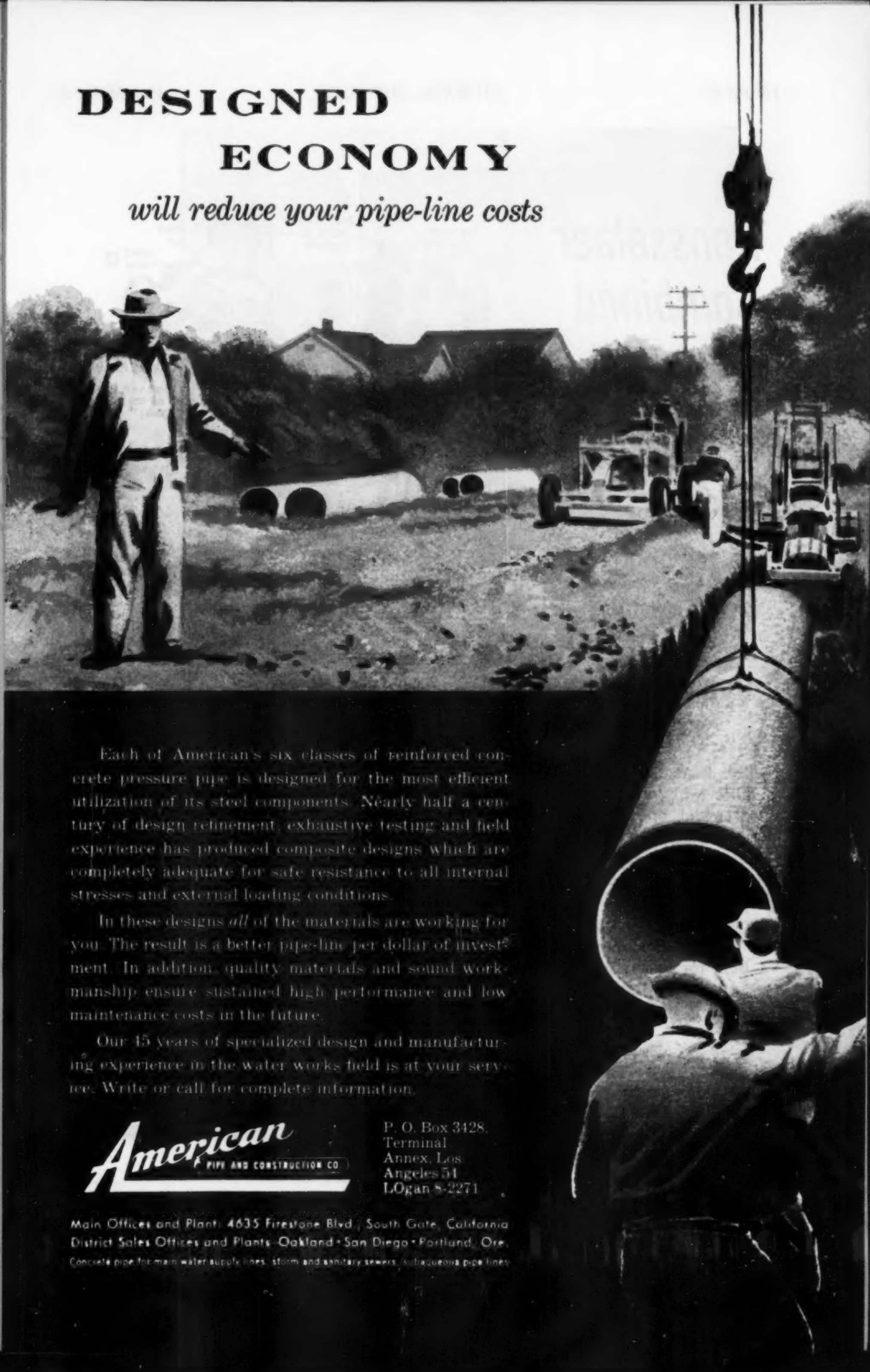
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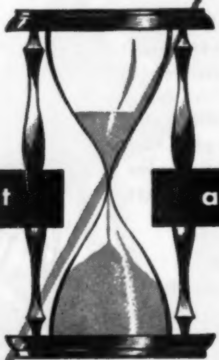

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Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 46 • FEBRUARY 1954 • NO. 2

Underground Movement of Bacterial and Chemical Pollutants

By R. G. Butler, G. T. Orlob, and P. H. McGauhey

A paper presented on Oct. 30, 1953, at the California Section Meeting, San Francisco, Calif., by R. G. Butler, Jr., Research Engr.; G. T. Orlob, Asst. Prof. of Civ. Eng.; and P. H. McGauhey, Research Engr.; all of Univ. of California, Berkeley, Calif.

WATER, more than any other single factor, limits the growth and controls the prosperity of the Southwest. The awareness of the people of this region of the dependence of their agriculture and industries on water is without parallel elsewhere in the United States. To water works engineers and officials, and to enlightened citizens alike, the idea of rejecting used water simply because it has acquired the unsavory name of "sewage" is becoming patently absurd. In what other field of human activity, they ask, is so valuable a vehicle employed to transport so small a burden; and in what other circumstance is the transporting vehicle discarded at the end of only a single journey?

The modern concept of water usage includes not only the collection, treatment, and distribution of potable water but also its re-collection, transportation, and retreatment after its use by the customer. The search for water is now being extended to what was formerly the aquatic junkheap. Ways are actively being sought to reclaim domestic and industrial waste waters. Some of the proposals that seem most economically hopeful at present involve returning these waste waters underground, taking advantage of the filtering ability of the soil and the diluting effect of ground water. The whole question of underground travel of pollutants, therefore, assumes a new importance.

The danger that public water supplies may become polluted as a result of the movement of bacteria and chemicals underground has long been a matter of concern to public health authorities. Laws presupposing such a dan-

ger have been enacted in most states for the purpose of protecting sources of water supply. California's law, for example, prohibits the discharge of any waters unfit for human consumption into underground water-bearing formations suitable for use as a source of domestic water supply. Of necessity, this and similar legislation have been based on incomplete scientific information, but the general welfare is served and any relaxation of regulations is unlikely until extensive investigations have shown clearly under what conditions waste waters may be returned to ground waters without endangering the public health.

Problems involved in reclaiming waste waters by adding them to existing ground waters include technological, economic, and psychological considerations, as well as the travel of bacterial and chemical pollutants. In addition, there are various legal aspects to the question of ownership of water returned to surface and subsurface storage at public expense. A discussion of the technological and economic problems is beyond the scope of this paper, although there is some possibility that they may prove to be the limiting factors because of the physical difficulties involved or the degree of pretreatment necessary. There is no reason, however, to expect that legal matters are incapable of resolution; and public displeasure with the ancestry of reclaimed waters should be easy to overcome. Since ancient times man has had faith in the inherent purity of cool spring and well waters. Furthermore, it has been demonstrated in various places that passing water through the soil dissociates it from any former status in the minds of people. In the West, there is ample reason to believe that citizens will accept reclaimed water whenever their health departments can give assurance of its safety. But, to give that assurance, more must be known about the underground travel of harmful bacteria and toxic chemicals.

Proposed Reclamation Methods

Cesspools and septic tanks have returned sewage effluents to the soil for centuries. Sewage farms and irrigation with sewage effluents have done likewise. Planned reclamation, however, is a more recent development, involving physical laws and fundamental problems of pollution travel that until recently have been generally unexplored and poorly defined. The most promising methods of reclamation in California include: [1] overirrigation of crops with waste waters, so that the excess becomes available for ground water replenishment by percolation through the soil; [2] spreading of waste water on the surface of the ground in special infiltration basins from which it may replenish ground water, again by percolation through the soil; and [3] recharging waste water directly into the ground water through injection wells penetrating underground aquifers.

For economic reasons, the pretreatment of the wastes in all three instances would be kept to the minimum required by practical considerations or by pollution travel, whichever might impose the limiting conditions. To define these conditions, investigations have been going on for more than 3 years under the sponsorship of the California Dept. of Health, the State Water Pollution Control Board, the University of California, and other groups. These studies are a part of a larger investigation covering both the feasibility of reclaiming waste waters

by returning them to the ground water and the ground water pollution potential of current waste disposal practices such as the use of refuse dumps and fills.

Two distinctly different aspects of pollution travel are involved in the three reclamation procedures mentioned, as well as in the leachings from refuse dumps, privies, septic tanks, cesspools, sewer wells, and polluted surface waters. These aspects are: [1] the movement of bacteria and chemicals downward with percolating water; and [2] the lateral movement of such pollutants once they have entered the ground water, either along with percolating water or by direct recharge.

Movement Above Water Table

Observers are generally in agreement that pollution is not appreciably extended laterally by percolating waters moving downward through soil above the ground water. The extent of vertical travel of pollution, therefore, becomes the more important factor in determining the public health danger involved in applying wastes of any given intensity of pollution to the soil and in defining the minimum safe distance between the ground surface and the water table. In this matter, different investigators have arrived at a variety of tentative conclusions.

In the early 1930's it was demonstrated in Los Angeles (*1*) that a highly polished sewage treatment plant effluent could be spread on soil without impairing the quality of ground water. A later survey by a board of engineers in Los Angeles County (*2*) concluded that safe additions may be made to the ground water. Favorable results were reported (*3*) from spreading sewage treatment plant effluent at Whittier and Azusa, Calif. A report to the Santa Clara, Calif., Water Commission (*4*) likewise concluded that a well polished effluent could be spread without producing significant changes in ground water quality. Based on these and similar observations, the general view is that reclamation by spreading is both safe and practical when the waste water undergoes a high degree of pretreatment.

Experiences with more highly polluted wastes are reported by several observers. Typical are the results of a study (*5*) of 50 pit privies in different soils during both rainy and dry seasons; no evidence of pollution was found in wells 25 and 40 ft from the pits. Soil tests indicated that pollution may extend only approximately 5 ft. Caldwell (*6*) concluded from his studies that coliform organisms move only 1–5 ft in dry or slightly moist soils. In the Dutch East Indies, however, investigators reported (*7*) finding coliform organisms in soil 2 years after contamination, in decreasing numbers down to a depth of 9–13 ft but increasing again as the ground water level was approached. In inhabited areas, sewage bacteria were found as deep as 33 ft, leading the investigators to conclude that purification in the soil is not as great as has been claimed.

Field Study

To learn more about sewage water reclamation, the University of California San. Eng. Research Labs. began a series of investigations of spreading, some of which are still in progress. The first study (*8*) was carried on at Lodi, Calif., under the sponsorship of the California Dept. of Public Health and the State Water Pollution Control Board. For a period of 28 months, water, primary settled sewage, and final

sewage plant effluent were applied to eight spreading basins, each 19 ft in diameter, surrounded by sheet-metal dikes. Four of these basins were equipped with center wells and sampling facilities, so that samples of the percolating liquid could be collected for bacteriological and chemical analyses at depths of 1, 2, 4, 7, 10, and 13 ft below the ground surface.

The surface layer of soil at the test site is a Hanford fine sandy loam. Underlying this is a coarser sandy loam, below which fine sand occurs down to 6–13 ft below the surface. The effective size and uniformity coefficient of the soil range greatly throughout this 13 ft of depth and vary widely between basins (Table 1).

Because the conditions leading to maximum percolation rates, as well as to minimum contamination of ground water, were of interest in the investigation, a number of operating variables were studied. In a typical study of pollution travel, sewage treatment plant effluent with approximately 10 ppm BOD was first applied to a basin for several months. Later, settled sewage with approximately 100 ppm BOD was applied to the same basin. Samples of percolating liquid were obtained at various depths and examined weekly. Table 2 shows the average results of the determination of the most probable number of coliform organisms in four of the spreading basins.

It is notable that a high degree of bacterial removal occurred near the surface in all instances and that changing from final effluent to the more highly polluted settled sewage did not result in increased penetration by coliform organisms. In Basins A and B the change actually resulted in decreased penetration, probably owing to a decreased percolation rate due to clogging of the soil surface by organic matter. Records show that the appearance of coliform organisms at a depth of 4 ft below the surface did not occur until 3–8 months after spreading had started.

A study of Table 2 reveals that, in Basins A and C, the average MPN at a depth of 2 ft exceeds the corresponding values at 1 ft below the surface. This observation was presumed to in-

TABLE 1

Effective Size and Uniformity Coefficient of Soil in Lodi, Calif., Spreading Basins

Depth *ft*	Basin A		Basin B		Basin C		Basin D	
	Effective Size *mm*	Uniformity Coefficient	Effective Size *mm*	Uniformity Coefficient	Effective Size *mm*	Uniformity Coefficient	Effective Size *mm*	Uniformity Coefficient
1	0.0056	45.5	0.0035	85.7	0.0032	67.3	0.0018	77.9
2	0.0019	116.0	0.0027	92.6	0.0025	96.0	0.0018	50.0
4	0.0036	66.7	0.0038	52.6	0.0014	111.0	0.0053	45.3
7	0.0420	11.9	0.0024	175.0	0.1550	3.2	0.0074	32.5
10	0.1200	4.3	0.1700	3.6	0.1600	3.1	0.0150	16.0
13	0.0800	7.9	0.0012	241.0	0.0026	25.4	0.0019	105.0
Avg	0.0422	42.1	0.0306	108.0	0.0541	51.0	0.0055	54.5

dicate the existence of channels in the soil which tended to lead surface water directly to the sampling pans at the 2-ft level. Exploration at the time the project was dismantled verified the existence of such local channels in both basins.

Figure 1 shows graphically the definite inverse relationship between sample depth and coliform count in Basin B. It illustrates that a bacteriologically safe water, as judged by US Public Health Service standards for drinking water (*9*), can be produced by percolating settled sewage or final effluent through at least 4 ft of soil like the Hanford fine sandy loam.

The results of the Lodi study, therefore, seem to substantiate the 5-ft distance considered reasonably safe by Kligler (*5*) and Caldwell (*6*), as well as the general conclusion that the addition of waste waters to the ground water may be bacterially safe. The data also indicated that, for the soil studied, bacterial disappearance in the percolating liquid is tremendously rapid in the first foot of soil; furthermore, the number of coliform organisms penetrating 1 ft or more, as measured by tests on the liquid, is essentially independent of the intensity of pollution of the waste water applied. This fact, together with an observed loss of percolation rates at the higher pollution intensity in settled sewage, suggested that the bacterial reduction was probably a surface phenomenon, resulting from clogging of the soil by organic matter, although no tests were made to show the incidence of coliform organisms in the soil itself at various depths. Assuming this explanation to be correct, it is inferred that large reductions in bacteria resulted from a filtering out of the organisms themselves and of the organic matter with which they were associated.

The study did not show the relationship between the incidence of coliform organisms in the soil and in percolating water at the same depth below the ground surface. Neither did it demonstrate whether a coarser soil might permit deeper penetration of coliform bacteria, because the high cost of field

TABLE 2

Coliform Organisms in Liquid Percolating Below Soil Surface

Depth *ft*	Avg MPN per 100 ml						
	Basin A		Basin B		Basin C	Basin D	
	Final Effluent Applied	Settled Sewage Applied	Final Effluent Applied	Settled Sewage Applied	Final Effluent Applied	Fresh Water Applied	Final Effluent Applied
0	179,000	4,140,000	188,000	5,700,000	188,000	232.0	164,000
1	1.2	1.6	482.0	20	148.0	1.6	0.2
2	285.0	32.0	5.6	0	305.0		
4	2.1	0.6	0.5	0	2.0		
7	0	0	0.2	0	0.2	0	0
10	0	0	0.1	0	0.1		
13			0	0	0.3	0	0

scale research made it impossible to repeat the Lodi experiments on other pervious California soils.

Lysimeter Study

To obtain further information on the phenomena involved in the movement of pollution with percolating water and, if possible, to generalize the experience at Lodi, the University of California, in July 1952, began a new series of pilot studies of spreading at its engineering field station in Richmond, Calif. Five types of pervious soils commonly occurring in California were placed in twenty lysimeters, each 3 ft in diameter and 5 ft in depth, so constructed that liquid would percolate through 3 ft of soil into a perched aquifer. Soils selected for study included the Hanford fine sandy loam used in the field study at Lodi and the Hesperia sandy loam investigated extensively in water-spreading studies by the US Soil Conservation Service at Bakersfield, Calif. In this manner, it was hoped to minimize the unknowns resulting from the unavoidable disturbance of soils for lysimeter studies, and to establish a correlation between field and pilot plant results that would permit extrapolation of lysimeter data to field conditions with some reasonable degree of accuracy. Table 3 shows the types and grain size characteristics of the five soils used.

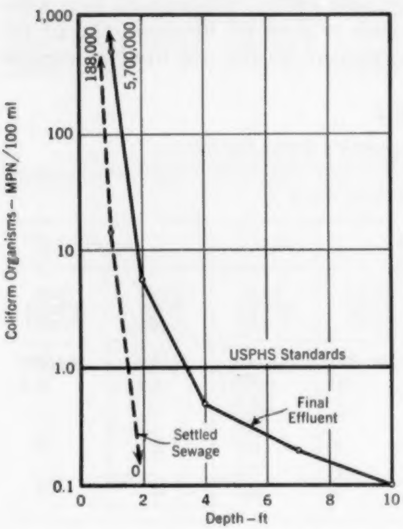

Fig. 1. Coliform Organisms in Percolant

The inverse relationship between sample depth and bacterial population is evident. Data are for Basin B.

Representative samples of each soil type were obtained in the field from the top 2 ft of material. After being thoroughly air dried, they were screened through 4-mesh hardware cloth and carefully placed in the lysimeters. Water was then spread on all units for a sufficient period to establish percolation characteristics and to ascertain that there was a satisfactory agreement between various lysimeters containing each soil type. Thereafter, settled sewage was applied to fifteen of the units to a depth of 1 ft, while the others were continued under fresh water as controls. Samples of the applied liquid and of the effluent from the units were analyzed bacteriologically and chemically each week for various periods up to 35 weeks. Bacterial penetration was determined from soil samples and from percolant samples at the same depth, where convenient.

Table 3 shows typical data on coliform organisms passing through 3 ft of each type of soil after 4 weeks of spreading with settled primary sewage; the sewage had an average BOD of 172 ppm and a coliform organism concentration of about 1.1×10^8 per 100 ml. It is evident that the bacterial

removal by soils of a relatively large effective size was much less than that by finer soils, while, in some soils, large amounts of bacteria could enter a ground water table existing at the 3-ft depth investigated. Of particular interest is the lack of correlation between the infiltration rate of various soils and the corresponding removal of bacteria by the soil. Other observations of infiltration rates show that, when sewage was first applied to the various soils, no measurable change in the rates established with fresh water occurred in the three tighter soils. On the Yolo loam, however, a decrease of 90 per cent occurred in 4 weeks; and a 99 per cent reduction took place within 24 hr on the Oakley sand. Thereafter the rates remained quite stable at the values shown in Table 3.

Investigation of the organic buildup within the soil showed that suspended organic matter was effectively removed from the infiltrating sewage within the first 0.5 cm of soil depth. A prolonged period of spreading, ranging from 11 to 22 weeks for various soils, did not produce a greater penetration of this organic mat. After 6 weeks of continuous spreading of sewage on the Oakley sand, the MPN of coliform organisms in the effluent dropped from a fairly constant value of 2.4×10^3 to approximately 230 per 100 ml, at which concentration it remained, with minor variations, throughout the remaining 7 weeks of study. During this entire 13-week period, the infiltration rate was fairly constant at approximately 0.1 ft per day. No similar pattern was exhibited by the Yolo loam, from which the coliform organisms per 100 ml of percolant fluctuated between 2.4×10^4 and 4.8×10^2, the tendency generally being toward the higher value. The other three soils all yielded coliform organism counts that varied little from the values shown in Table 3 from week to week, except that an occasional extremely high count appeared, which may be of great significance should further studies prove it to indicate a periodic unloading of organisms in the soil.

TABLE 3.

Lysimeter Study Data

Soil Type	Effective Size *mm*	Uniformity Coefficient	Infiltration Rate *ft/day*	Coliform Organisms in Percolant MPN/*100 ml*
Hanford fine sandy loam	0.0056	26.7	0.3	less than 45
Hesperia sandy loam	0.0020	75.0	0.2	less than 45
Columbia sandy loam	0.0034	51.4	0.3	less than 45
Yolo sandy loam	0.0155	8.4	0.3	24,000
Oakley sand	0.015	14.6	0.1	2,400

The quick development of an organic mat in the top 0.5 cm of all soils and the failure of this mat to migrate to greater depths meant, of course, that a large number of bacteria intimately associated with the solids must be removed at the soil surface. The relative effectiveness of this organic mat and the underlying soil in reducing the coliform organism concentration in the liquid from the initial MPN of 1.1×10^8 to the values shown in Table 3 and similar studies could not be evalu-

ated solely from the results of bacterial examination of liquids reaching the perched aquifers in the lysimeters. Therefore, a series of studies of coliform organism counts in both soil and liquid at various depths was undertaken. Results show that a limiting zone is slowly built up in the soil, its depth below the surface depending upon the

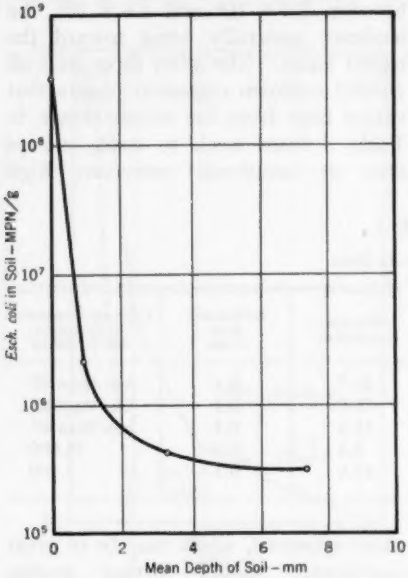

Fig. 2. Coliform Organisms Near Surface

The graph shows the typical average MPN of Esch. coli *per gram (dry weight) of soil within the top 8 mm.*

nature of the liquid applied and the surface treatment of the soil. Under various operating conditions studied, this zone occurred at 10–50 cm below the soil surface and did not seem related to particle size. A substantial reduction in bacteria in the percolating liquid occurred as the liquid passed through the limiting zone.

In one instance, in which sewage with an average MPN of 1.1×10^8 was applied to Hanford fine sandy loam, the percolating liquid still carried 8.4×10^7 organisms when it reached a depth of 47 cm below the surface. At 64-cm depth, however, the count had been reduced to only 230 organisms per 100 ml. Tensiometers showed that between these two depths a steep hydraulic gradient existed, indicating the presence of a limiting zone.

Observations of coliform organism counts in the soil itself produced evi-

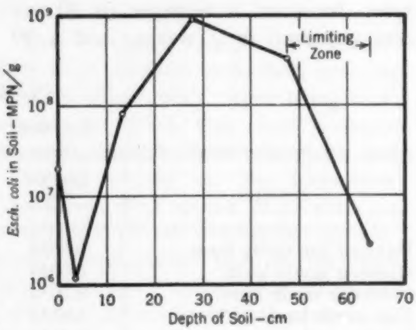

Fig. 3. Coliform Organisms in Hanford Soil

The MPN of Esch. coli *per gram (dry weight) of Hanford soil within a 64-cm depth is shown.*

dence that the bacteria are drastically reduced in numbers through the surface mat in the first 0.5 cm of soil, but that a subsequent buildup of *Esch. coli* occurs at lower levels. This fact can be accounted for only if organisms are filtered out of the percolant at a rate greater than their death rate within the soil. These phenomena are illustrated graphically in Fig. 2 and 3.

Figure 2 is typical of all five soil types investigated. The particle size

of soil had no apparent effect on the thickness of the surface mat formed or on the MPN of organisms found at various depths within the mat. From Fig. 3, it is notable that the buildup of organisms in the Hanford soil reached counts greater than that of the applied sewage. Other studies showed that this was not true in the coarser Oakley sand.

Table 4 shows a comparative organism count in Hanford and Oakley soils at various depths, together with the MPN of the final percolant. The maximum buildup of organisms in the Hanford soil reached a value of more than 9.1×10^8 at a depth of 25 cm below the surface. Although the Oakley sand reached its maximum at the same depth, the density of organisms was only 8.9×10^5. In both soils, the applied sewage contained an MPN of 1.1×10^8 *Esch. coli.* The final percolants showed a different pattern, the Hanford percolant having 230 *Esch. coli* per 100 ml in contrast to the 6×10^3 coming through the Oakley sand. From these and similar observations, it may be concluded that the removal of bacteria from liquid percolating through a given depth of soil is inversely proportional to the particle size of the soil.

Although it is not possible to correlate MPN per 100 ml of percolating liquid with MPN per gram of soil, it is evident that a lightly contaminated effluent may emerge from a grossly polluted soil. The contaminant potentialities of this polluted soil in the event that the water table rises to intersect it have not been satisfactorily explored. A careful study by Stiles and Crohurst (*10*) revealed a tendency for pollution to stay in the capillary fringe of ground water when the water table lowered. This might be expected in fine soils like the Hanford, Hesperia, and Columbia types investigated. In a coarser material it is possible that pollutants picked up in rising ground water coming in contact with polluted soil might travel laterally with ground water flow in accordance with laws somewhat different from those involved in the downward travel of such material with percolating water. Studies on coarse soils like the Oakley sand, however, have by no means shown that pollution of the soil is more serious than pollution travel in the percolant. They indicate that a 3-ft depth, even though a limiting zone develops, is not as effective as a similar depth of finer soil in removing coliform organisms from the percolant. The fact that the Oakley soil was a disturbed sample can be somewhat discounted by the reasonably good agreement between the results of bacterial-removal studies on undisturbed Hanford soil at Lodi and the same soil in lysimeters. The gen-

TABLE 4

Coliform Population in Hanford and Oakley Soils

Depth *cm*	*Esch. coli* in soil—MPN/g*	
	Hanford	Oakley
1.0	1.2×10^6+	6.8×10^5
4.0	1.2×10^6	6.0×10^4
		†
12.5	8.4×10^7	2.6×10^5
25.0	9.1×10^8+	8.9×10^5
47.0	3.42×10^8	
	†	
65.0	2.84×10^6	6.9×10^5
Percolant	230/100 ml	6×10^3/100 ml

* Dry weight.
† Denotes approximate location of limiting zone as shown by tensiometers.

eral belief is that, at some reasonable depth, coliform organisms will effectively be removed from percolating waste water, but the need for further studies with deeper columns of coarse soils is clearly indicated.

Chemical Movement

The movement of dissolved chemicals with percolating water differs, of course, from the travel of suspended bacteria. Where clay is present, cation exchange will take place until an equilibrium is established between the soil and the applied liquid. Studies of travel of chemicals through Hanford soil at Lodi (*8*) showed that calcium, magnesium, sodium, and chlorides remained relatively constant through the 13 ft of soil depth observed. Sulfates, bicarbonates, and nitrates increased very markedly. Phosphates disappeared within the first foot, and potassium decreased by approximately 50 per cent below 7 ft. Similar information resulted from the lysimeter studies on different soils, although there was a variety of important reactions that are more pertinent to water reclamation problems other than those of public health. It seems evident, from available information, that chemical changes in percolating sewage are not great and that many applied chemicals can be expected to reach the ground water along with percolating liquids. This should not be a serious consideration, however, unless the sewage contains toxic materials or unusually high concentrations of such undesirable elements as chlorides, sulfates, sodium, or boron.

At Lodi, the BOD of percolating liquids did not exceed 5 ppm, even at 1 ft below the surface, when the applied BOD was 100 ppm. In the lysimeters, however, where the applied BOD averaged 171 ppm, a different situation developed: in all instances, the BOD of the percolant passing through 3 ft of soil varied with the length of time the sewage was applied. Beginning at approximately 5 ppm at the end of 1 day, the BOD steadily increased to a value of approximately 100 ppm at the end of 4 weeks. Thereafter a decline set in, which showed evidence of leveling off at the end of 25 weeks, at which time the typical BOD was approximately 20 ppm, regardless of soil type. These data show a progressive decrease in penetration by unstable dissolved solids but indicate that there are conditions under which a relatively high BOD may be carried into the ground water.

Pollution Travel With Ground Water

Reports of the travel of pollution with ground water movement reveal a somewhat imperfect knowledge of the conditions under which lateral travel of bacterial and chemical pollutants might occur. Investigators seem to agree that pollution travels farthest in the direction of ground water flow and that chemical pollutants travel much farther than bacterial pollutants. Stiles and Crohurst (*11*) showed this to be the fact in the movement of water from sewage-polluted trenches. In a sand of 0.13-mm effective size, bacteria traveled 65 ft in 27 weeks, while chemicals traveled 115 ft in the same period. These researchers (*10*) compared the movement of coliform organisms with that of the chemical, uranin, from polluted trenches intersecting the ground water. They found bacteria 232 ft and uranin 450 ft from the trench. In both studies (*10, 11*) they reported movement in the direction of ground water flow only and more extensive travel in wet weather than in dry.

Ditthorn and Luerssen (*12*) report the results of introducing *Bacillus prodigiosus* into an aquifer of 32.8 per cent porosity at a point 69 ft from a well. The bacteria appeared in the well on 10 consecutive days, beginning with the ninth day, and were found as long as 30 days after injection had ceased. When great numbers of the organisms were applied above the ground water table, none appeared in a well 58 ft distant.

A series of studies in which latrines were bored 3–5 ft into the ground water was conducted by Caldwell (*13–16*). In one location, where the effective size of the soil was 0.08 mm, coliform organisms penetrated 10 ft and anaerobes 50 ft, while chemical pollutants were observed 300 ft down the stratum. Another latrine, penetrating an aquifer in which water was moving 10–16 ft per day, was lined with perforated boards supporting fine soil. Test wells 10 ft away showed no coliform organisms, although odors, foaming, and pH changes were observed. In one pit, extending 3 ft into ground water moving 13.3 ft per day, coliform organisms traveled more than 80 ft but later regressed to 20 ft. This phenomenon was observed in another pit, in which the initial rate of flow of pollution from the latrine approached the ground water velocity but then receded as clogging developed (*13*). In this instance, bacteria traveled 35 ft and chemicals 90 ft.

Activated sludge effluent, traced (*17*) from percolation beds to a spring 1,500 ft away, passed through fine sand in a narrow stream. Coliform organisms were absent after 400 ft, but iron bacteria flourished in the spring. Ammonia dropped from 12 ppm to 6 ppm in 1,400 ft, while nitrates increased from 0.04 to 10 ppm.

A few reports indicate little travel of bacteria with ground water. Meinzer (*18*) suggests that the travel of bacteria seems limited in sands. Sampson (*19*) reported in 1934 that wells 150 ft from percolation beds produced sterile water. In Germany, Austen (*17*) found bacteria disappearing within a few yards in seepage moving at a velocity of approximately 3 ft per day.

Chemical Travel

As previously mentioned, chemical pollutants travel farther than bacterial and have caused many European, especially German, ground water supplies to be abandoned. Such chemicals include wood-tar residues, which traveled 197 ft; picric acid wastes, traveling several miles; and pickling liquors, traveling an unspecified distance. Lang (*20*) reported that leachings from an old garbage dump reached wells 1,476 ft away, causing an increase in total solids from 360 to 552 ppm, and in hardness from 190 to 272 ppm. In 1940 Lang and Bruns (*21*) noted a picric acid waste travel of 3 miles in 4–6 years. Wells 2,000 feet downstream from cooling ponds showed a temperature rise and an increase in manganese, hardness, and iron. In other instances, garbage dumped in a sand pit continued to pollute wells 2,000 ft away 15 years after the dumping had ceased, and chlorinated sewage from leaking pipes caused phenol tastes and fungus growth in wells 300 ft away. Dye added to the sewage traveled 300 ft in 24 hrs. Rossler (*22*) recently observed an increase, after 10 years, in chlorides, hardness, and manganese in wells below a garbage dump. Austen (*23*) records the pollution of wells in Breslau, Germany, by seepage from a river 165 ft away and reports tests

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showing artificial recharge to be productive of changes in the chemical composition of well water, notably in iron and hardness.

Similar data have been observed in the United States. At Vernon, Calif., chemical contamination has traveled 3–5 miles (*24*). In Michigan (*25*), chromate wastes have advanced through sand to pollute wells at a distance of 1,000 ft in 3 years. Caldwell (*17*) found chemical pollution traveling 47 ft in a width of 25 ft and a depth of 7 ft in ground water moving only 0.2–1.5 ft per day. Calvert (*26*) reported an increase in hardness, calcium, manganese, total solids, and carbon dioxide in wells 500 ft from an impounding pit for liquor from a garbage reduction plant. Sayre and Stringfield (*27*) found phenol wastes traveling 1,800 ft in ground water in one instance and failing to penetrate 150 ft in another.

The movement of salt brines with ground water seems especially pronounced. When 800 kg of sodium chloride was placed in a sand pit, the salt soon reached a well 233 ft away. Sumps containing oil field brine (*28*) contaminated ground water so that wells ¼ mile away became unfit for use in irrigation. Salt placed in a cesspool (*29*) reached a well 200 ft away in 24 hr.

Recharge Well Study

To help clarify the chaotic picture of pollution travel with ground water movement, the University of California San. Eng. Research Labs., in 1951, began a research study under the sponsorship of the California Water Pollution Control Board. Installations for recharging degraded water into an underground aquifer and observing the resulting travel of chemical and bacterial pollution include a system of 24 6-in. inspection wells surrounding a 12-in. recharge well, all penetrating a 5-ft thick pressure aquifer located approximately 100 ft underground. Pipelines and equipment have been installed to make it possible to pump raw sewage to a settling tank, mix the tank effluent with water, deliver the mixture to the recharge well site, and introduce it underground at a controlled rate. Suitable equipment is provided for sampling the observation wells and redeveloping the recharge well. Because of the extent of the necessary installations and the multiplicity of problems involved, conclusive information is not yet available.

In one experiment, primary settled sewage diluted with fresh water was injected into the aquifer at a rate of 32 gpm for a period of 41 days. The injected liquid had a suspended-solids content of 4.0 ppm, a BOD of 3.3 ppm, and a coliform organism MPN of 1×10^6 per 100 ml. Bacterial pollution was observed to travel in the direction of ground water movement at approximately half the velocity of tracer chemicals, reaching sampling wells 100 ft south and 50 ft east of the recharge well in 33 hr. The coliform die-away and dilution was so rapid, however, that the MPN reaching these sampling wells never exceeded 38 organisms per 100 ml of water. During the 41-day period of injection, no increase in coliform organism count developed and no organisms reached sampling wells 225 ft south and 100 ft east of the point of injection. Additional studies of the underground movement of injected pollution are planned as soon as mechanical difficulties have been overcome.

Summary and Conclusions

The movement of bacterial and chemical pollutants with water percolating through soil above the ground

water table has been studied somewhat more extensively than the travel of pollution with ground water movement, although much remains to be investigated in both fields. From reports in the literature and from the results of field and pilot scale studies being conducted by the University of California San. Eng. Research Labs. on soils continuously spread with settled sewage or sewage treatment plant final effluent, and on underground recharge, a number of conclusions can be drawn:

1. Two distinctly different aspects of pollution travel may be involved in waste water reclamation and in leachings from septic tanks, cesspools, refuse dumps, and the like: [1] the movement of bacteria and chemicals with percolating water above the ground water table; and [2] the movement of bacteria and chemicals with ground water.

2. The movement of percolating water, and, consequently, of bacteria and chemicals in soils spread with sewage waters, is vertical unless ground water is reached before travel ceases.

3. It has been reported that bacteria will travel little more than 5 ft in moist or dry soils. This statement seems to be true for fine soils under continual submergence with sewage. There is evidence, however, that it may not apply in coarse soils.

4. Where the ground water table is near the surface of a spreading basin, it must be presumed that coliform bacteria will enter the ground water through such means of access as porous soil, root channels, or rodent holes.

5. Under soil conditions like those observed in a Hanford fine sandy loam at Lodi, Calif., a bacteriologically safe water can be produced by percolating primary or secondary sewage treatment plant effluent through at least 4 ft of soil.

6. When sewage is spread on Hanford fine sandy loam under field conditions, the number of coliform bacteria found in the percolant 1 ft below the soil surface is not appreciably affected by the intensity of pollution applied.

7. Bacterial removal in any given depth of soil is appreciably less for soils of relatively large effective size than for finer soils.

8. There seems to be no correlation between the infiltration rate into various types of soil and the removal of bacteria during percolation of water through them.

9. Within the range of soil types observed, the removal of suspended organic matter is effectively accomplished in the first 0.5 cm of soil depth, regardless of the effective soil size.

10. There seems to be no tendency for the organic material collecting at the soil surface to migrate downward with time.

11. The organic mat at the surface of the soil removes a large number of bacteria associated with suspended solids. The principal removal of suspended bacteria, however, occurs in the soil in some limiting zone through which there is a steep hydraulic gradient.

12. A greater number of the coliform organisms which have passed the limiting zone remain in the liquid at any given depth in a coarse soil than in a fine soil.

13. Bacterial buildup occurs in the soil, owing to the long survival time of *Esch. coli* in comparison with percolation rates. This buildup may far exceed the intensity of pollution applied at the surface and is directly proportional to the reduction of bacteria in the percolant.

14. Various observers have confused soil pollution with pollution of the percolant. Actually, lightly contaminated effluents may emerge from highly pol-

luted soil. The ability of the polluted soil to contaminate a rising ground water table later is not known.

15. The chemical nature of percolating sewage is little altered by its passing through as much as 13 ft of Hanford soil after the initial ion-exchange equilibrium has been satisfied, except for those substances that undergo progressive changes through bacterial action.

16. Applied chemicals can be expected to reach the ground water along with percolating water, but, with adequate dilution in the ground water, the result should not be serious unless toxic elements or high concentrations of certain undesirable materials have been added to the waste water.

17. There are conditions under which a relatively high BOD may be carried into ground water with percolating water.

18. Various observers report confirmed instances of travel of coliform organisms with ground waters through distances of 10–232 ft.

19. Chemical pollutants travel farther and faster than do bacterial pollutants in the ground water. The literature reports chemical travel over distances varying from a few feet to several miles, and from 2 to 30 times as far as coliform bacteria introduced at the same time.

Acknowledgment

Many individuals contributed to the investigations on which this report is based. Grateful acknowledgment is made to members of the California Dept. of Public Health for assistance in designing the studies and to the sanitary engineering faculty of the University of California for advice and counsel in carrying them out. The Lodi studies were conducted by Vinton W. Bacon, Raymond V. Stone, and Arnold E. Greenberg; and the recharge studies, by Raymond V. Stone and Ray B. Krone. Jerome F. Thomas supervised the laboratory analyses. All investigations were under the direction of Harold B. Gotaas, Director, San. Eng. Research Labs., Univ. of California, Berkeley, Calif.

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Water Spreading Operations in the San Gabriel Valley

By Finley B. Laverty

A paper presented on Oct. 30, 1953, at the California Section Meeting, San Francisco, Calif., by Finley B. Laverty, Chief Hydr. Engr., Los Angeles County Flood Control Dist., Los Angeles.

TO those who have grown up with the water business in Southern California, the spreading of water in the upper portion of the San Gabriel Valley is an accepted practice of long standing. Irrigation interests have conserved the normal flow of the San Gabriel Canyon by maintaining a 500-acre spreading ground near its mouth since 1916. During the past 17 or 18 years the Los Angeles County Flood Control Dist. has, as one of its assigned duties, conserved flood waters through spreading grounds, first in the stream bed area above Whittier Narrows, later in an offstream location a short distance below the narrows, and recently in the upper fringe areas such as the Raymond Basin. The development and operation of these grounds, at a time when the ground waters of the coastal plain were being overdrawn by tremendous industrial and residential growth, have given rise to consideration of the use of this method to store imported waters in the large reservoirs created by the underground aquifers.

This discussion will briefly outline spreading ground development; some of the experience gained; potential use of the grounds; and some factors involved in supplying the ultimate demands of the area and, at the same time, saving the coastal ground water basin from destruction by sea water intrusion.

Description of Area

As shown in Fig. 1, San Gabriel Canyon, with an area of more than 210 sq miles of mountain drainage, is the principal mountain region producing flows that traverse the stream beds of the San Gabriel Valley. East and west of San Gabriel Canyon, an additional 84 sq miles of mountain area is also tributary to this valley. This section, however, consists mostly of such small drainage areas that the mountain water production is absorbed either by direct diversion or by percolation in small ground water basins before it reaches the main San Gabriel Valley. In fact, as indicated above, the normal runoff of the San Gabriel Canyon is diverted for use or conserved before it reaches the lower valley area. Flood regulation structures, however, have created means of storing and releasing storm flows at rates that may be handled by water conservation facilities. These flood control structures are: Cogswell Dam, in the west fork of the San Gabriel Canyon; San Gabriel Dam, below the canyon forks; a portion of the storage behind the Metropolitan Water District's Morris Dam, located a few miles above the mouth of the canyon;

and the US Army Corps of Engineers' Santa Fe Reservoir, approximately 4 miles below the mouth of the canyon. In addition, the Corps' Whittier Narrows Reservoir, now under construction, will bridge the gap between the hills that define the boundary between the so-called San Gabriel Main Basin and the Central Coastal Basin to the south.

The Santa Fe Reservoir overlies the deep, coarse alluvial fill of the San Gabriel Main Basin and is, in itself, a spreading area. This ground water basin, extending from Foothill Boulevard to Whittier Narrows, is, in fact, a tremendous water storage reservoir that may be likened to a saucer, so tipped that a continuous outflow of rising water issues at Whittier Narrows. The quantity of rising water is directly dependent upon the elevation of the water table at the key well in Baldwin Park. The central part of this underground water basin, that is, the part directly affected by the percolation of flows of the San Gabriel River, has an average storage capacity of approximately 6,500 acre-ft per foot of depth, over a range of 50 ft below and above the water table as it existed in January 1933. (The water table then was 10 ft above the present level.) Between Whittier Narrows and the Inglewood-Newport ground water barrier, the coastal plain comprises a deep, alluvium-filled valley, where, because of the distance from the mountains, the sediments are much finer than in the San Gabriel Main Basin. Along the course of the Rio Hondo and the San Gabriel River, the first 5 miles below Whittier Narrows consists almost entirely of fine, clean sands and gravels, making a good percolating medium. This area is termed the Montebello Forebay, as percolation there serves to supply the aquifers that underlie the clay cap extending from the forebay to the Inglewood-Newport uplift.

During the period of record for these two basins, the Main Basin water table has varied from 49 ft to 140 ft below the ground surface at Baldwin Park, a range of 91 ft; in the Montebello Forebay area, the water table has ranged from a few feet below ground surface to a Sep. 7, 1953, low of 111 ft, as measured at the key well midway between the spreading grounds.

In the pressure area, there is a record for a well that was developed in 1895 with an artesian flow sufficient to rise 80 ft above ground surface. Water levels in the vicinity of this well are currently (Jun. 30, 1953) the lowest on record, 70 ft below sea level.

Studies by the US Geological Survey and the Los Angeles County Flood Control Dist. have shown that sea water is being drawn into the Coastal Basin across the Inglewood-Newport fault through Dominguez Narrows, as well as at the Alamitos Bay gap. This salt water intrusion has already rendered some private wells useless and is encroaching on some of the water supply wells of Long Beach.

Spreading Ground Types

Before discussing the conservation studies and the major works in the San Gabriel Valley, it may be of interest to note the sequence and types of spreading ground development.

As mentioned previously, a 500-acre spreading ground has been operated since 1916 at the mouth of San Gabriel Canyon, utilizing flows from a power conduit that discharges at the mouth of the canyon. At the time of construction of these grounds, land had little value; hence, this entire area was developed with canals and ditches

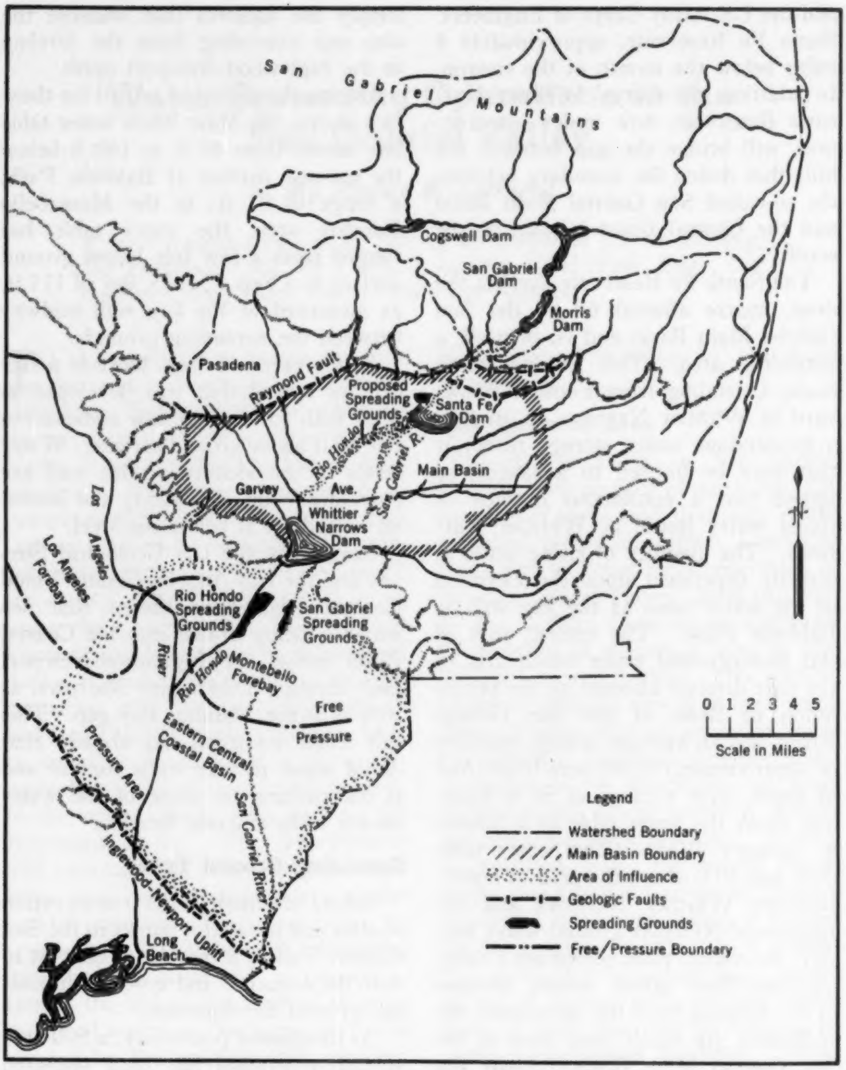

Fig. 1. Basin Boundaries and Spreading Grounds

The stippling outlines the portion of the Central Coastal Basin influenced by water conserved below Whittier Narrows. The "free/pressure boundary" marks the approximate transition from free ground water to ground water pressure levels.

to percolate a flow of 90 cfs. When the Los Angeles County Flood Control Dist. started its extensive spreading ground developments in the 1930's, the encroachment of subdivisions and industry on the wash areas and the con-

sequent increase in the price of land posed the problem of developing spreading methods that would require a minimum area to conserve the available regulated storm waters. Therefore, instead of the long-tried ditch-and-furrow method of spreading, attempts were made to utilize canals serving basins several acres in extent and percolating water from a depth of 2–3 ft of storage.

2 shows the arrangement of a typical temporary spreading ground intake structure. In the stream bed adjacent to the intake is a spillway designed to handle surplus spreading or storm water commensurate with operating conditions. The rest of the stream bed is diked off by a low earth fill that washes out if storm flows exceed the capacity of the intake and temporary spillway.

Fig. 2. Temporary Spreading Ground Intake Structure

A spillway adjacent to the diversion canal intake is designed to handle surplus spreading or storm water. The rest of the stream bed is diked off by a low earth fill that washes out if storm flows exceed intake and spillway capacity.

Experience was also gained in the use of gravel pits by developing inlet chutes and studying the relation between the percolation in pits 30–50 ft deep and that in the shallower basins.

To get water to offstream spreading areas, temporary diversion works were constructed, similar to normal irrigation diversion—that is, with a side-channel inlet capable of handling sufficient flow to fill basin storage and maintain the percolation rate. Figure

This type of temporary diversion has been used successfully for most of the district's spreading grounds, both below regulating reservoirs and below unregulated mountain canyons. An example of the latter is shown in Fig. 3, which pictures the temporary spreading ground development at the mouth of the Arroyo Seco, near Pasadena. The spreading grounds are above Devils Gate Reservoir, where the formerly porous bed was sealed by

mud flows as a result of mountain fires.

Figure 4 shows the use of gravel pits below Eaton Wash Debris Dam. The pit, which provides a percolating capacity of approximately 5 cfs, is approximately 700 ft long and 150 ft wide, and averages 25 ft in depth. This and one other pit were used in 1952 to conserve more than 1,100 acre-ft in the adjudicated Raymond ground water basin where pumping rights sell at $39 per acre-foot—the price set by the overall cost of reimbursing Pasadena for its pumping rights with Metropolitan Water Dist. water.

Until recently, practically all of the Los Angeles County Flood Control Dist. diversions to offstream spreading grounds were accomplished by means of the type of structure shown in Fig. 2. The completion of concrete flood channels, however, has now made possible the construction of permanent headworks, with radial gates, up to 60 ft in length, forming the diversion barrier to create pools from which gates in the side wall of the channels control the discharge to the spreading ground canals. The costs of these permanent diversion structures vary from $80,000 for Tujunga Wash to more than $400,000 for the Rio Hondo Coastal Basin spreading grounds.

Fig. 3. Arroyo Seco Spreading Ground Development

The photograph shows a downstream view of the temporary spreading grounds located near Pasadena, Calif.

Water Resources Studies

During the early 1930's the water conservation section of the Los Angeles County Flood Control Dist. hydraulic division worked with consultants Haehl and Etchevery (*1*) in a study of the potential salvage of flood waters that would be afforded by the creation of a series of offstream spreading grounds to be utilized in conjunction with San Gabriel Canyon reservoirs. These studies, completed in 1936, were based on an estimated ultimate annual demand of 253,000 acre-ft in the Main and Coastal basins. It was considered that works capable of spreading 500 cfs in the Main Basin and an equal amount in the Coastal Basin from available flood water supplies would be very nearly capable of meeting the ultimate demand.

Because little was known of percolation rates and the best means of developing large spreading grounds, Etcheverry and Haehl recommended that spreading works be established gradually and modified as experience was gained. Initial development during the middle 1930's consisted of a series of test basins and spreading channels in the wash area of the Main Basin from the split of the San Gabriel and Rio Hondo to Garvey Avenue on the San Gabriel, and in the stream bed of the Rio Hondo to above Santa Anita Wash.

From this operation, it was found that a consistent percolation of 10 ft in depth per day could be obtained by basin spreading in the area now covered by the Santa Fe Reservoir, utilizing canyon water for periods of 1–2 months. Further downstream in the Main Basin, where the sediments were finer, the percolation rate decreased to 3–4 ft per day (*2*). This experience made it clear that the canal-basin method of development would result in the most economical use of any offstream spreading areas where land costs were important. The basin type of development makes it possible to cover at least 80 per cent of the gross area purchased, whereas the ditch-and-furrow method seldom gave percolation contact for more than 15 per cent of the area utilized.

Fig. 4. Eaton Wash Spreading Grounds

The inlet chute supplying water from Eaton Wash Debris Dam and the clean gravel walls of the pit make possible a percolation rate of 5 cfs.

The next endeavor, in 1938, was to develop 40 acres of offstream spreading grounds on the east side of the Rio Hondo below Whittier Boulevard. These grounds, together with a 60-acre development in 1939 on the San Gabriel below Whittier Boulevard, were enlarged in steps. Thus, only a small portion of the present grounds was available for spreading the generous yield of the mountain watersheds during the wet years from 1938 to 1944.

The gross area of canals and basins on the Rio Hondo (Fig. 5) is now 443 acres and that on the San Gabriel is 111 acres. (These areas make up a considerable portion of the 1,809 acres of spreading grounds of the Los Angeles County Flood Control Dist.)

Although the recent dry years have not afforded many opportunities to obtain further data on the percolation capacity of these 554 acres of spreading grounds, together with that of adjacent stream channels, it is considered that these works can handle 1,000 cfs of percolation to the underground water basin as long as the water table is more than 50 ft below the surface, the capacity decreasing to approximately 450 cfs when the water table is within 25 ft of the surface.

Fig. 5. Rio Hondo Coastal Spreading Grounds

The gross area of canals and basins on the Rio Hondo is now 443 acres.

Since the completion of the Etcheverry and Haehl studies, the California Div. of Water Resources has reviewed the San Gabriel Valley ground water supply on three occasions: a report (*3*) based on data available prior to 1941 was prepared in connection with the water rights applications of the San Gabriel Valley Protective Assn. (this SGVPA study was reviewed by a report (*4*) of the Los Angeles County Flood Control Dist. in 1943); State Div. of Water Resources Bulletin No. 53 gave an evaluation of water resources as of 1944–45 (*5*); and State Water Resources Board Bulletin No. 8 brought the overdraft determination for the Central Coastal Basin up to 1949–50 (*6*).

Table 1 indicates the surplus or overdraft in the Main and Central Coastal basins as of the date of the information used. As there is no overdraft in the Main Basin, these figures indicate that the supply potential for the Central Coastal Basin has changed from a surplus of 47,000 acre-ft in 1926 to an overdraft of 77,000 in 1950, a total change of 124,000 acre-ft. This growing depletion of storage indicates

the possibility of increased use of the Central Coastal Basin as a fluctuating storage reservoir to meet the demands of the overlying industrial and residential communities.

Bulletin 53 was developed at a time (1945) when the draft was sufficiently small so that only by pumping the Montebello Forebay down to 120 ft below ground surface at the previously mentioned key well during dry periods could all the regulated flood waters of the following wet period be spread without bringing the water table within damaging distance of the ground surface. A corollary to this statement would be that only small amounts of imported water could be spread for short periods during a succession of dry years; otherwise, the forebay basin would be so full that flood waters would have to be wasted during the ensuing wet period.

When, however, seven consecutive dry years (1944–51) occurred, coinciding with a large increase in draft, the problem of continued water supply for the Central Coastal Basin became acute. Wells went dry and pumps were lowered; organizations were formed to consider methods of supply; the Central Basin Municipal Water Dist. was established; and the area will ultimately hold an election in order to become a part of the Metropolitan Water Dist. of Southern California. The water level is now 111 ft below ground surface and is dropping at an increasing rate.

As a temporary stopgap during this quest for water, it was suggested that a zone might be formed within the Los Angeles County Flood Control Dist. to purchase untreated Colorado River water from the Metropolitan Water Dist. for spreading below Whittier Narrows, in order to reduce the rate of ground water decline. This proposal led to current studies to determine the potential of the spreading grounds as a means of offsetting the basin overdraft.

From available records, it was possible to calculate the supply of ground water necessary to maintain the Montebello Forebay ground water levels at a given elevation, thereby determining the outflow from this forebay pool to the remainder of the Central Coastal Basin ground water body underlying the clay cap. Additional data from state and district sources led to the conclusion that the ultimate average annual overdraft of the Central Coastal Basin would approach 110,000 acre-ft, as compared to 77,000 during 1949–50.

TABLE 1
Surplus or Overdraft in Main and Central Coastal Basins Combined

Reference	Date of Data	Surplus (+) or Overdraft (−) *acre-ft*
Bul. 7 (*7*)	1926–27	47,000 (+)
SGVPA study (*3*)	1940–41	28,400 (+)
Bul. 53 (*5*)	1944–45	22,000 (−)
Bul. 8 (*6*)	1949–50	77,000 (−)

An important portion of this ultimate overdraft which was not considered in the Haehl-Etchevery report (*1*) is the estimated ultimate export to the ocean of more than 56,000 acre-ft of sewage and industrial waste water annually from the San Gabriel Main Basin and Montebello Forebay, in addition to 14,000 acre-ft originating in the Raymond Basin. A large portion of this waste was formerly available to the Montebello Forebay as rising water or surface stream flow from sewage treatment plants, or as direct replenishment to the ground water body from domestic cesspools. Another important factor in the rapidly increasing overdraft is the high rate

of increase of hard-surfaced areas, due to paving and building in subdivided sections overlying formerly pervious formations. As compared with the combined Main and Central Coastal Basin average annual demand of 253,000 acre-ft given by Haehl and Etchevery in 1936, the figure is now 320,000. It is estimated that the demand will level off at 390,000 acre-ft by 1980.

Taking into account the data on outflow from Montebello Forebay and ultimate draft, studies of the expected average annual water supply have led to some conclusions radically different from those which it was possible to obtain at the time Bulletin 53 was completed. The current conclusions are based primarily on the consistently lower water table inherent in the present and ultimate draft from the Central Coastal Basin. They are:

1. There are three sources of water supply for the Central Coastal Plain: (a) percolation from local rainfall and local storm waters; (b) Owens River Aqueduct water, utilized as a surface supply by the part of the basin within Los Angeles city; and (c) Colorado River water from the Metropolitan Water Dist. Aqueduct, available either as a direct supply of treated water or from the underground basins after the spreading of untreated water.

2. Approximately two-thirds of the basin supply is derived from wells and delivered through independent water systems; hence, replenishment of the local supply of ground water furnishes the most widely effective means of meeting distribution needs without considerable capital investment in pipelines and surface storage.

3. The spreading of an adequate supply of imported water in existing spreading grounds on the Rio Hondo and San Gabriel River below Whittier Narrows will benefit water levels in the entire eastern portion of the Central Coastal Basin pressure area, as well as in the Montebello Forebay area where the water is introduced. Benefits to the lower part of the basin may be retarded by the time required to refill some of the pervious stringers that have been drained during the extreme lowering of the water table in this area. Therefore, it may be advisable to supply the lower part of the basin by pipeline with Colorado River or Forebay water while the ground water levels recover. The only other section of the basin where the water level would not immediately benefit from spreading in the Montebello Forebay is the East Los Angeles–Vernon area, in which there is an extreme draft. Fortunately, this area is adjacent to the trunk lines of the Metropolitan Water Dist., and it would be easy to replace pumpage with a direct supply of Colorado River water.

4. Contrary to findings in 1945, when Bulletin 53 was written, current and ultimately anticipated conditions indicate that water levels in Montebello Forebay can never reach such an elevation that a series of wet years would cause the loss of percolation of local storm waters. In fact, it seems doubtful whether, without imported-water replenishment, the ground water level in Montebello Forebay can ever again rise to within 35 ft of the surface.

5. The underground storage capacity created by the present draft on the basin would permit a sizable amount of imported or reclaimed water to be stored in the basin, in addition to the maximum salvage of local mountain and valley storm water.

6. Detailed studies indicate that, whenever the autumn elevation of

ground water in the Montebello Forebay is more than 60 ft below the surface, 25,000 acre-ft of imported water can be spread annually without the loss of local water conservation even if the succeeding winter rainfall is more than 200 per cent of normal.

7. An autumn elevation of more than 70 ft below the ground surface will permit the spreading of 50,000 acre-ft under the same condition. (The present depth is 111 ft.)

8. Irrespective of water table elevation, approximately 27,000 acre-ft of imported or reclaimed water could be stored underground every year without reducing the salvage of local flood water more than an average of 3,000 acre-ft per year.

From these findings, it seems apparent that the underground storage basin of the Central Coastal Plain can serve as a regulating reservoir to store sufficient imported or reclaimed water to supply 33–50 per cent of the ultimate annual overdraft of approximately 110,000 acre-ft. The remainder of the overdraft can undoubtedly be met by gravity use of imported water, first from the Colorado River and then from sources proposed in the California Water Plan (*8*).

Fortunately the location of the district's spreading grounds below Whittier Narrows makes it practicable to supply them by obtaining untreated Colorado River water or by erecting a sewage reclamation plant adjacent to the San Gabriel Valley trunk sewer that parallels the Rio Hondo spreading grounds. Either of these sources of supply can readily produce at least 25,000, and possibly 50,000, acre-ft per year of usable spreading water. Hence, economics and permanence of supply are perhaps the prime factors in deciding the best way to meet the Central Coastal Plain overdraft.

With the Central Basin Municipal Water Dist. a part of the Metropolitan Water Dist. of Southern California, it may be expected that Colorado River water could be supplied to the spreading grounds at no more than $10 per acre-foot. As the proportion of the total supply of Colorado River water available to a given unit of the Metropolitan Water Dist. depends on the assessed value of that unit in relation to the others, the capital structure of the Central Coastal Plain would seem to assure a continually adequate supply of water, both for spreading and for gravity delivery of treated water to those portions of the region not immediately benefited by additions to the ground water basin.

Supply of the overdraft through construction of a sewage reclamation plant involves a considerable capital outlay. It has been estimated that the cost of water delivered to the spreading grounds, on the basis of 50-mgd capacity and a 50-year investment retirement period, would be approximately $13 per acre-foot. Consequently, unless the Colorado River supply is curtailed beyond expectation, it appears less costly than reclaimed water. Another disadvantage of the reclaimed-water supply is the continued permissive addition of industrial wastes to the sewage system of the San Gabriel Valley. Many of these wastes would not present difficulties to a sewage reclamation plant, but, if the chemical content of the water were altered to the extent that it could not safely blend with natural ground waters, pollution control authorities would certainly prevent its use.

Summary

The foregoing discussion has outlined some of the Los Angeles County Flood Control District's experience in

the development of offstream areas to conserve flood waters. Perhaps the highlight of this experience is the devising of techniques of concentrating on areas overlying pervious sands and gravels, resulting in at least an 80 per cent beneficial use of the property set aside for water conservation. Of course, a corollary of this method is careful attention to maintenance of the grounds and employment of the minimum personnel for their operation. Experience indicates that the capital investment and operating cost for spreading grounds 150 acres and larger would not exceed $8.00 per acre-foot of flood waters conserved. It has also been determined that, if the development costs of the grounds are assigned to the initial purpose of conservation of flood waters, then the total spreading cost of conservation of imported waters will not exceed $3.00–4.00 per acre-foot. Thus, the total cost of imported water stored in the ground water basin amounts to, say, $13 per acre-foot, the equivalent tap cost being less than $20. Such a water would have a quality advantage, having been blended with local ground water.

This paper has discussed an example of a large ground water basin where the considerable overdraft creates such a severe lowering of the water table as to permit replenishment largely by means of storage of imported water in the basin through spreading. The use of imported and reclaimed water has been touched on, with the conclusion that, where the former is available to a political unit because it is already paying the costs of membership in the Metropolitan Water Dist., this source of water is likely to provide the cheapest permanent supply for meeting the ground water basin overdraft.

The benefits and difficulties of supplying all of a ground water basin's pumping requirements from a single spreading location have been described. It has been noted that some sections of the basin, particularly the more distant industrial areas, will be hard to supply with ground water to meet pumping drafts during a succession of dry years. Such areas are best accommodated through surface distribution lines directly from the imported supply. To prevent the loss of a portion of the ground water basin through sea water intrusion, pumping must be decreased to reverse the landward gradient of the saline water.

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Control of Sand in Water Systems

By John R. Rossum

A paper presented on Oct. 29, 1953, at the California Section Meeting, San Francisco, Calif., by John R. Rossum, Engr. of Sanitation, California Water Service Co., San Jose, Calif.

SAND is frequently a problem to the operator of a system supplied from wells. The abrasive action of sand causes severe wear of pumps, chlorinator injectors, and valves. By abrading or clogging consumers' fixtures, sand will contribute materially to water waste and lead to complaints. Sand clogging the orifice of a control mechanism may cause the failure of important equipment or may stop meters and thereby decrease revenue. Excessive sand production from a well indicates the formation of cavities, which may eventually result in the collapse of overlying strata and damage the well. Experiments have shown that the accumulation of sand in pipes may greatly increase the friction loss.

There is little in the literature regarding the control of sand except for discussions on the design of sand traps using Stokes' law or its modifications relating settling rate to particle size. The engineer faced with the problem of designing a sand trap may employ these formulas to compute the dimensions of a trap that will remove, for example, 95 per cent of all particles having a diameter of 0.2 mm, but this is of little value unless it is possible to determine the sand concentration in the supply and to establish the quantity that can be tolerated in the distribution system.

As sand grades imperceptibly into gravel on the one hand and into silt on the other, any precise definition of the term must include arbitrary and artificial limits on particle size. For the purpose of this discussion, such a definition is undesirable and unnecessary, because physical conditions impose an upper limit (the size of rock particle that can be pumped from a well), while the lower limit is roughly established by the implication that the particles can be removed by a conventional sand trap.

The collection of a representative water sample for sand determination has been given careful consideration. A sample collected from a horizontal pipe carrying water at low velocity is unsatisfactory. It has been demonstrated that representative samples can be obtained from vertical pipes, and fairly satisfactory samples may be taken from horizontal pipes if the flow is highly turbulent. Samples taken immediately downstream from ells, tees, and other fittings that create turbulence are generally satisfactory if the water velocity is 5 fps or more. The water velocity through the sampling tap at the point of diversion from the main stream should be at least as great as the velocity in the main stream.

The concentration of sand in water from any given well varies with operating conditions. This fact complicates the problem of sampling, for, if a sample is to be representative of the aver-

age sand concentration, it must be taken over a considerable period of time, during which the flow through the sampling device must be reasonably constant; hence, the design of the sand collector must be such that it will function properly with infrequent attention. On the other hand, it is often desirable to take samples over a very short period of time. For this reason, the sand sampling device must be capable of handling a sufficient volume of water in a few minutes to permit the separation of a sand sample of adequate size.

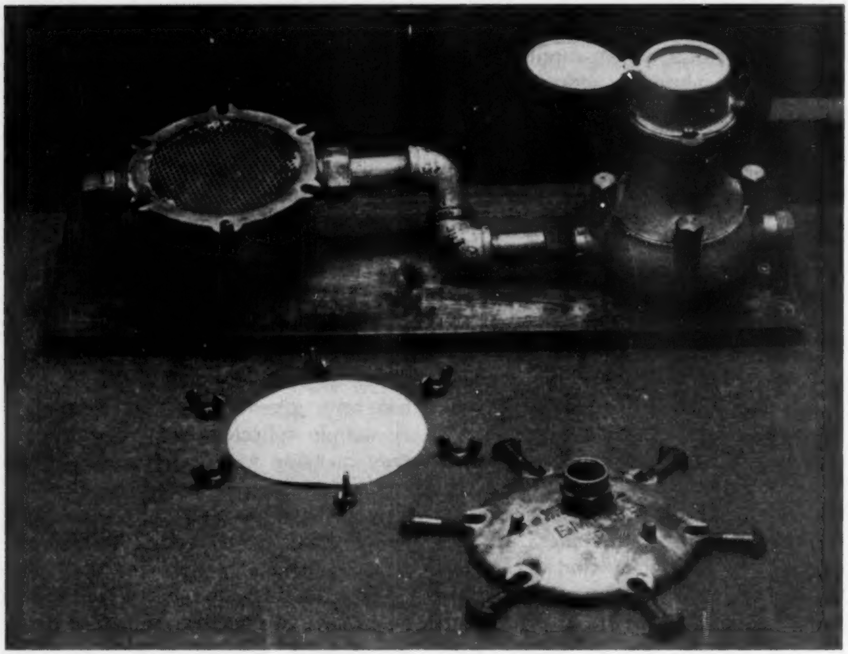

Fig. 1. Filter Collector

The mercerized batiste filter cloth is supported on a perforated disk 5 in. in diameter. The filter frame is made from a meter case.

Sand Collectors

A review of the literature in the field uncovered very little previous work on the quantitative measurement of sand. Hamel (*1*) illustrates a sand content indicator. In a communication to the author, Hamel has described the use of this device. In effect, a 5-gal bottle serves as a settling basin to collect the sand from a known volume of water. The quantity of sand collected is measured and its concentration computed. Bunker (*2*) describes a method in which a 2-liter sample is collected, swirled, and allowed to settle, the resulting sand disk being compared with standards of known concentrations.

Trials of numerous devices for collecting sand samples, including those of Hamel and Bunker, led to the conclusion that, for samples collected over

Fig. 2. Centrifugal Sand Sampler

The water enters tangentially, and the entrained sand, thrown to the side by centrifugal force, falls into the tube. The sand-free water flows out through the hole in the baffle.

a short time interval, filtration was the most practical and convenient method. For continuous sampling, a centrifugal sand separator was found to be superior to other types considered.

The filter device is illustrated in Fig. 1. Essentially, it consists of a cloth filter followed by a meter. The filter frame, fabricated from a meter case, supports the filter on a perforated disk with a diameter of approximately 5 in. The cloth is firmly held between gaskets, and wing nuts facilitate its insertion and removal. Water enters through a hose connection on top and flows through the filter to the meter

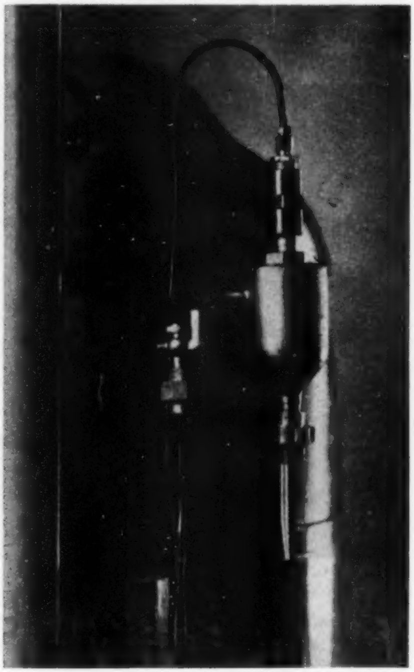

Fig. 3. Centrifugal Sampler in Operation

A control valve maintains the flow at a constant value, regardless of the inlet pressure.

and thence to waste. The cloth used is mercerized batiste, selected because it can be purchased in almost any dry goods store, has about the right porosity, and is reasonably uniform. Where the water to be sampled is under a pressure of approximately 50 psi, 50 cu ft may be sampled in less than 20

min. It is seldom necessary to collect samples larger than this.

The cloth filter retaining the sample is sent to the laboratory, along with a report giving the size and location of the sampling tap, pertinent data on well operation, and the volume of water sampled. Upon arrival in the laboratory, the sand is dried, its volume measured, and its concentration computed. It may then be subjected to particle size or settling rate analysis, if desired.

The filter type of sand sample collector is limited to applications where it can be attended continuously. As soon as an appreciable quantity of sand is collected, the head loss through the filter increases and the quantity of water flowing through the machine decreases to such an extent that the velocity at the point of sampling is too small to assure a representative sample. This difficulty is overcome by the centrifugal type of sand sample collector.

Christensen (*3*) discusses the design principles of centrifugal sand separators. Figure 2 illustrates a sand tester that embodies these principles and is adapted to the purpose in hand; Fig. 3 shows the unit in operation. Water enters the body of the device tangentially immediately below the baffle. The small radius and high velocity create a large centrifugal acceleration, which throws the sand to the side of the device. The sand falls down the side and is collected in the centrifuge tube, while sand-free water flows out through the hole in the center of the baffle. The flow is maintained at a constant value, independent of the inlet pressure, by means of a flow control valve * that is rated at 0.5 gpm.

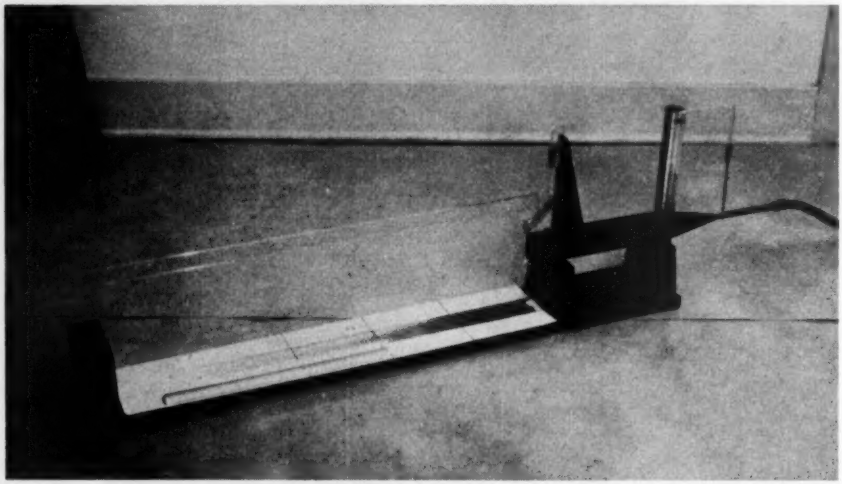

Fig. 4. Settling Rate Determination

This device uses compressed air to separate sand samples into groups of particles of uniform fineness. A diagram of the equipment is shown in Fig. 5.

* Manufactured by Dole Valve Co., Chicago.

At suitable intervals, the volume of sand collected is recorded, together with the number of hours of operation. From these data, the average sand concentration may be computed, as the flow through the tester is known.

The patented flow control valve contains a rubber orifice that contracts with increasing inlet pressure in order to maintain a constant flow. According to the manufacturer, it is designed for a pressure variation from 15 to 150 psi. The author's firm has tested the device over a pressure range of 20–120 psi and has found less than 10 per cent variation in the flow, although the actual flow may be as much as 20 per cent above or below the rated value. The length of useful life of this valve is not known to the author.

The use of a centrifugal sand tester on a well permits sand production to be recorded daily. Any significant increase is noted immediately and corrective action can be taken before appreciable quantities of sand have entered the distribution system. The low cost of these testers makes it feasible to provide one for each well that is suspected of producing excessive sand.

Settling Rate Determination

While sampling procedures were still in the experimental stage, it became obvious that screen analyses were unsatisfactory for determining settling rates, because the sand usually contained mica. Methods for establishing the settling rate in water directly were tried, but they were all so cumbersome and inconvenient that they were abandoned in favor of an air method. A photograph of the equipment built for the purpose is shown in Fig. 4, and a diagram is given in Fig. 5. Compressed air enters at a rate controlled by the needle valve. This rate is easily reproduced by means of a pressure gage and an orifice. Straightening vanes reduce the turbulence of the air stream. Sand from the hopper im-

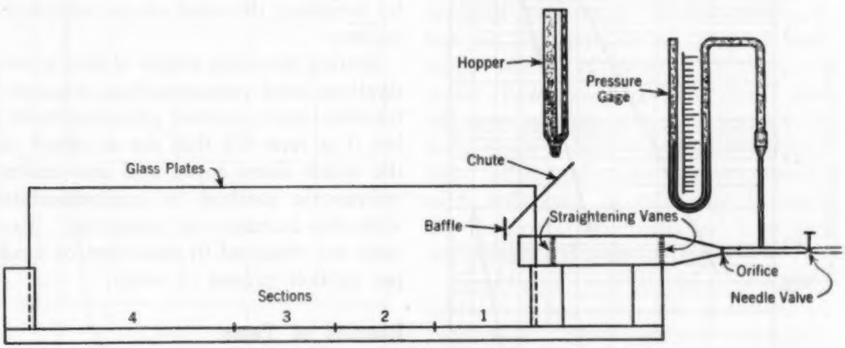

Fig. 5. Separator Layout

Sand from the hopper falls into a compressed-air stream flowing between two glass plates. The coarsest, most rapidly settling particles are deposited on Section 1, the finer particles being carried to the other sections. The range of settling rates in water for sand in each of the sections has previously been determined.

pinges on the baffle and falls into the air stream as a thin ribbon. The sand-laden air is prevented from flowing laterally by the parallel glass plates. The most rapidly settling particles fall on Section 1, while the finer and more slowly settling particles are carried progressively to Sections 2, 3, and 4.

The air flow is initially established and the machine calibrated by trial. A representative sand sample is run through the device at several different air rates, and individual particles are collected from each section. These particles are dropped, one at a time, through the column of water in a liter graduate, and their sedimentation rates are determined with a stopwatch. It was found convenient to adjust the air blast so that the sand falling on Section 1 had a settling rate of more than 7 fpm; that on Section 2, 3–7 fpm; Section 3, $2\frac{1}{2}$–3 fpm; and Section 4, less than $2\frac{1}{2}$ fpm.

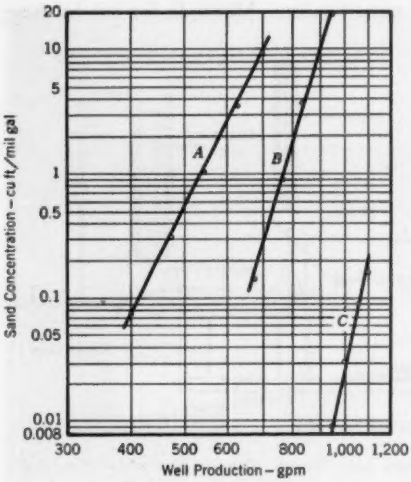

Fig. 6. Well Production and Sand Concentration

Values of K *for Curves* A, B, *and* C *are, respectively, 539, 772, and 1,194; values of* n *are, respectively, 8.6, 14.5, and 21.3.*

In using the machine, the sand sample is first dried and its volume determined in a centrifuge tube. The air blast is turned on and adjusted to its proper value. The sand sample is then poured into the hopper. After the sand has been graded, the volume falling on each section is determined. The total sand concentration and the percentage of sand in each group are reported. If the sample is less than 1 ml, satisfactory results can be obtained by weighing the sand on an analytical balance.

During the early stages of this investigation, sand concentrations were determined and reported gravimetrically, but it is now felt that the accuracy of the much more rapid and convenient volumetric method is commensurate with the accuracy of sampling. Results are reported in cubic feet of sand per million gallons of water.

Results of Tests

The use of sand testers has yielded some interesting and significant information on sand production in wells. Experience with sand testers has so far been limited to wells drilled in recent alluvial formations where the aquifers consist of relatively thin layers of sand or gravel. Small testers have been installed on gravel envelope wells, perforated-casing wells, and open-bottom wells.

When sand production for an individual well is measured at various rates of flow, the concentration of sand varies exponentially with water production. In making these tests, sufficient time must elapse between each measurement to permit the sand concentration to reach a steady value. When these results are plotted, the

points fall on a smooth curve represented by the equation:

$$S = \left(\frac{Q}{K}\right)^n$$

in which S is the sand concentration, in cubic feet per million gallons of water; Q is well production in gallons per minute; and K and n are observed constants. The value of n has been found to vary from approximately 5 to

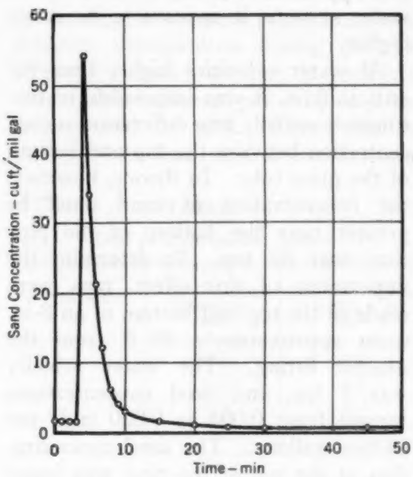

Fig. 7. Concentration at Start of Pumping

After the first few minutes of operation, the sand concentration falls sharply and levels out.

more than 20 but is commonly approximately 15. Thus, a small change in water production makes a large change in sand production.

The data for three wells are shown in Fig. 6. Similar results have been obtained from tests on six additional wells. Three of these fit the exponential equation very well, and two of them exhibit a few erratic points that are attributed to inaccurate data; in the other well, sand production ceased when the pumping rate was increased above a certain flow. In this exceptional instance, it is believed that all the sand was coming from the uppermost set of perforations and that, when the pumping rate increased so that the water level dropped below this set of perforations, sand production stopped.

Sand concentration is considerably greater when a well is first started than it is after a period of continuous operation. This effect is illustrated in Fig. 7.

It was hoped that sand testers would permit the evaluation of the various types of sand traps. Unfortunately, it has not been possible to compare sand traps under identical conditions of water flow, sand concentration, and effective particle size. A number of tests made under varying conditions have failed to demonstrate any significant difference in efficiency between the horizontal sedimentation type and the centrifugal type of trap.

Sand testers were found to be highly useful in controlling the operation of sand traps. They permit a reasonably accurate calculation of the volume of sand accumulated in a sand trap, so that it may be flushed when a predetermined percentage of the total volume is filled with sand. Prior to the use of sand testers, many traps were flushed much more frequently than necessary, sometimes with detrimental effects because opening the blowoff valve on the trap resulted in a lowered discharge head on the pump. This caused a substantial increase in water production, which, as previously mentioned, led to a very large rise in the rate of sand production. Occasionally wells produced more sand in the few minutes during and immediately after the flushing of a sand trap than in 24 hr of normal operation. When a well is equipped with a tester, sand concen-

trations may be observed during flushing, and either the rate of flushing or the well production may be throttled to meet the exigencies of the situation.

Sand in Pipelines

It has been found that, even after action has been taken to prevent excessive sand from entering the distribution system, complaints about sand may persist for a considerable period. Adequate sand control measures must provide for the removal of sand from the distribution mains in addition to the prevention of further accumulation. In order to carry out an effective flushing program, the behavior of sand in pipes and in networks of pipes was studied by direct observation in the laboratory and by indirect methods in water systems. The literature on this subject deals with single grains of sand (*4*) or with higher concentrations than are normally found in water (*5*).

The hydraulic transportation of sand through pipes is a highly complex phenomenon. The large number of variables involved has impeded both theoretical and experimental investigations. Basically, at very low water velocities, the sand does not move. As the velocity is increased, the sand collects on the bottom of the main, forming hills that are similar to the ripples so often noted on sandy stream beds. These ripples have a flat slope on the upstream side and a comparatively steep slope on the downstream side. The flow of water rolls sand grains up the flat slope to the crest of the ripple, whence they fall down the steep slope. The net result of this process is an apparent motion of the ripples in the same direction as the water flow, at a rate that is very slow compared with the average water velocity.

As the flow of water is increased, it reaches a critical velocity at which the sand is picked up in the highly turbulent area immediately downstream from the crest of the ripple. The numerical value of the critical velocity depends upon the size, shape, density, and concentration of the particles; the diameter and roughness of the pipe; and the temperature of the water. In small glass tubes, the critical velocity was approximately 1 fps, but in actual water systems it appears to be much higher.

At water velocities higher than the critical flow, it was impossible to distinguish visually any difference in concentration between the top and bottom of the glass tube. In theory, however, the concentration of sand must be greater near the bottom of the pipe than near the top. To determine the importance of this effect, taps were made in the top and bottom of an 8-in. main approximately 30 ft from the nearest fitting. The water velocity was 5 fps, and sand concentrations ranged from 0.004 to 0.020 cu ft per million gallons. The sand concentration at the top of the pipe was found to be approximately 70 per cent of that at the bottom, and the sand from the top showed appreciably more fines.

As might be expected, the flow of sand in distribution systems is vastly more complicated than in straight tubes. In order to study the behavior of sand in pipe grids, a network of glass tubes was employed. These laboratory observations were supplemented by studies of sanded meters and consumer complaints.

The most significant phenomenon observed in the glass model occurred when the water flow rates were adjusted so that the sand flowed in ripples as previously described. When one of these ripples travels down a

large tube and passes an intersection where a smaller tube tees off, a relatively low flow in the side tube will disrupt the ripple and a disproportionately large fraction of the sand will enter the side tube. This observation indicates that more sand troubles should be found in the smaller mains. Studies of sanded meters and consumer complaints confirmed this assumption, as is shown in Table 1.

The data for sanded meters and consumer complaints were taken from two different communities during periods arbitrarily chosen so that, in each instance, the incidence per mile of 2-in. main would be 10.0. These studies also indicate that sand difficulties are somewhat more prevalent on dead-end mains, and there seems to be a tendency for them to concentrate in low spots in the distribution system, but the effects are secondary and difficult to evaluate.

TABLE 1

Sand in Distribution System

Pipe Size *in.*	No. of Sanded Meters per Mile	No. of Consumer Complaints per Mile
2	10.0	10.0
4	1.3	2.7
6	1.1	1.1
8	0.1	0.6

Flushing of Mains

When establishing flushing programs to alleviate sand troubles, it is natural to place emphasis on flushing the smaller mains, where the bulk of the complaints originate. As the behavior of sand in the distribution system became better understood, it was realized that the sand complaints from the small mains were merely evidence that sand was accumulating in the large mains supplying them. The first practical test of this theory was spectacularly successful. Sand troubles had developed in a portion of a distribution system supplied from a number of wells. Sand testers showed that only one well was responsible for the troubles. This well was immediately throttled, and a continuous record indicated that sand production was effectively controlled. Sand troubles in the affected area continued undiminished, however, even after several months. The frequency of flushing of small mains in the area was increased but had very little salutary effect. Sand complaints were received at an average rate of approximately five per day. As soon as load conditions permitted, a 12-in. blowoff was installed on the 12-in. main delivering water from the well that had been producing sand. After flushing, no sand complaints were received for 3 days. During the next 30 days, only six complaints were recorded from the area. This main is now flushed annually, and complaints are rare.

Unfortunately, such satisfactory results cannot always be achieved. It is sometimes impossible to find locations where the considerable flow of water required to flush large mains adequately can be disposed of. Flushing velocities should be at least 5 fps, and preferably much higher. The total volume of water should be approximately ten times the volume of the main. Thus, with a velocity of 5 fps, a main should be flushed for 3⅓ min for each 100 ft of length. On a 12-in. main 1,000 ft long, the required flow would be a minimum of 1,800 gpm for a period of 33 min, a total of 60,000 gal. Where it is not possible to get rid of the entire quantity of water needed, it is more effective to flush at the required velocity for a shorter period

than it is to reduce the velocity. It is doubtful if a flushing velocity of less than 5 fps is of much value.

Measures Adopted

On the larger mains, sand complaints may sometimes be eliminated by putting the consumer's tap on top of the main. On small mains, this affords no relief, but a considerable reduction in sand may be obtained by feeding these mains vertically from the top of larger mains. Such practices may alleviate isolated conditions, but it must be remembered that the sand avoided by these consumers will stay in the system until others are troubled, or, less likely, until it is removed by flushing.

To make full use of the sand testers, it has been necessary to set a limit on the permissible concentration of sand. Studies have been made on the relation of consumer complaints and sanded meters to sand production in different communities. Approximately 80 wells have been investigated with somewhat conflicting results. In view of the difficulties encountered in making such studies, erratic results are not surprising. The great time lag between sand production and its ill effects must be given due consideration. In one study, it appeared that sand progressed from zone to zone at a rate of approximately 1 mile in 6 months. The size and character of the sand grains may also influence the tolerable concentration. Nevertheless, where the sand concentration is less than 0.10 cu ft per million gallons, no sand troubles have been found, while, where it exceeds 0.3 cu ft, consumer complaints or sanded meters invariably result. A purely arbitrary limit of 0.1 cu ft per million gallons has been tentatively selected.

This investigation has led to the adoption of a routine practice for controlling sand. Centrifugal sand testers are installed on all wells delivering water to the afflicted area, in order to establish which wells are excessive sand producers. Sand concentrations are reduced by: [1] changing from intermittent to continuous operation; [2] curtailing water production; or [3] installing a sand trap. The proper choice of method may be facilitated by further tests using the filter type of sand sampler; if a sand trap is required, the settling rate must be determined. After controlling sand production from the wells, a study of the distribution system is made to determine where sand may have accumulated and to find a means of eliminating it from the system.

Consumer complaints, which were previously on the increase, have been reduced approximately 50 per cent since this program has been in effect. It is believed, however, that the incidence of clogged meters, which has been reduced by 82 per cent, is a better criterion of the effectiveness of the measures adopted.

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Pneumatic Controls and Instrumentation for Water Plants

By Russell H. Babcock

A paper presented on Sep. 28, 1953, at the Missouri Section Meeting, Excelsior Springs, Mo., by Russell H. Babcock, Asst. Mgr., Utilities Industry Div., Foxboro Co., Foxboro, Mass.

A NUMBER of recent water treatment plant designs have incorporated pneumatic control rather than hydraulic-mechanical types. Cost reductions have been realized in original equipment, installation, and maintenance. Engineers in other fields have used pneumatic control for a long time, some flow control installations having been in service for more than 20 years. Tens of thousands of such installations are at present in operation. Because of the interest of water engineers, it seems advisable to review the fundamentals of pneumatic control.

As measurement is essential to control, a knowledge of the principles of flow measurement is necessary to an understanding of the methods of flow control. Flow measurement in water treatment plants is normally accomplished by using one of the various primary devices, such as the orifice plate or venturi tube, to produce a differential pressure that is a function of the velocity of flow in the pipe. Through the proper selection of the size of the orifice and the range of the differential detecting device, meters can be built to read directly in terms of flow.

The controllers are usually remote from the point of measurement, making it necessary to transmit the measurement to the controller. Pneumatic transmission is the standard used throughout industry and will be discussed later.

A control system is made up of three separate parts: the measurement mechanism, the controller, and the mechanism controlled. In measuring flow, the measurement mechanism, which is usually incorporated with the transmission mechanism, could be a mercury manometer with transmitter or one of the various nonmercury transmitters. The controller could be directly connected to the measuring device—for example, a mercury manometer mounted on the back of a controller—or a pneumatic receiver-controller might be used. The controlled mechanism, which receives the air signal, would, in this instance, be a valve.

The measurement is made and transmitted to the controller; the controller, in turn, transmits a signal to the controlled mechanism so as to maintain the measurement at the desired control point; the new measurement is transmitted to the controller; and so on. Thus, automatic, continuous correction of the valve is provided in order to maintain the variable (the flow rate) at the desired value. This control circuit is not peculiar to pneumatic instruments but applies regardless of whether the system is pneumatic, hydraulic, electric, or mechanical.

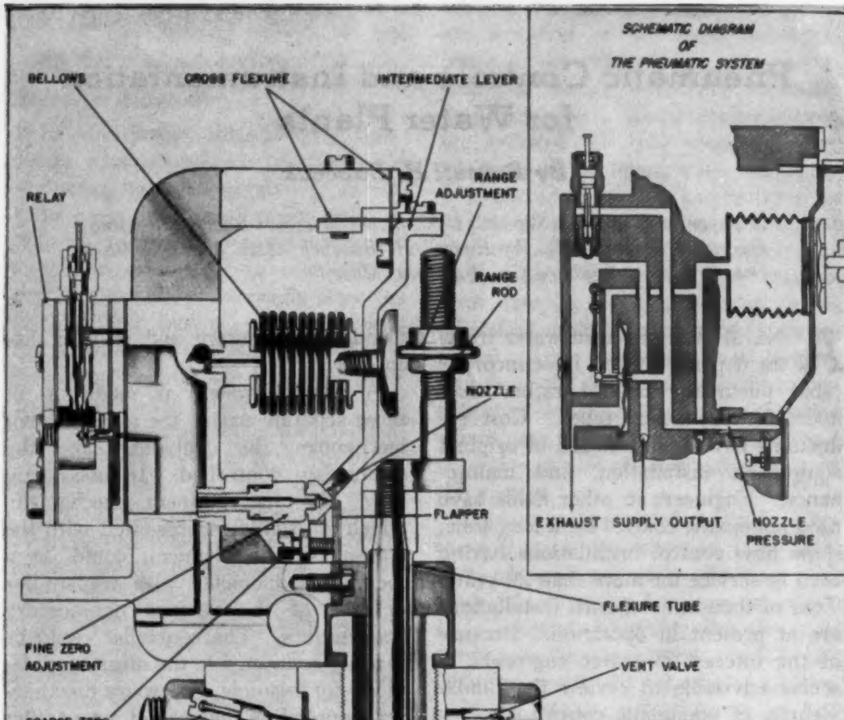

Fig. 1. Pneumatic Flow Transmitter

A cutaway drawing of a force-balance differential-pressure transmitter is shown.

Pneumatic Mechanisms

The Instrument Society of America has adopted a range of 3–15 psi as standard for outputs from transmitters and controllers; that is, both pneumatic transmission signals and control signals must fall within this range. Pneumatic instruments for transmitting differential measurements have a signal of 3 psi at zero differential and 15 psi at maximum differential. For example, if there are two pneumatic transmitters, one with a range of 0–25 in. of water and the other with a range of 0–200 in. of water, they both transmit a signal of 3 psi at zero differential and a signal of 15 psi at their respective maximums of 25 or 200 in. of water. Pneumatically operated valves of the air-to-open type would be closed when the signal to the valve was 3 psi and open when the signal was 15 psi. At any signal pressure between these two extremes, the valves would assume an intermediate position proportional to the pressure. The establishment of this standard permits the use of a pressure element calibrated from 3 to 15 psi as a receiver, regardless of the range of the pneumatic transmitter. It allows almost unlimited combinations of transmitters, receivers, controllers, and valves.

An example of the pneumatic flow transmitter is the force-balance differential-pressure transmitter shown in Fig. 1. Any difference between the pressures causes the diaphragm to exert a proportional force on the lower end of the force bar. With the flexure tube acting as a fulcrum, the range rod, or upper end of the force bar, imposes a proportional force on the flexure-pivoted intermediate lever.

Any tendency toward motion by the intermediate lever is immediately detected by the flapper nozzle, which alters the air pressure in the bellows so that it exerts a force on the intermediate lever exactly balancing that of the range rod. The intermediate lever is thus maintained in a fixed position, and the pressure in the bellows on one side is exactly proportional to the force imposed by the range rod on the other. The pressure element of a receiver connected to the bellows measures the force the bellows is imposing, which is proportional to the differential pressure across the diaphragm. This is the basic principle used in most differential-pressure transmitters of the force-balance type.

Pneumatic controllers consist basically of a measurement mechanism and set point, with a means of detecting and correcting errors between the measurement and set point. Various follow-up mechanisms fulfill additional control functions. Depending on the type of controller, it is possible to provide exact point control, control within a specified band, or on-off control. Flow controllers are usually of the proportional reset type, providing control at a point. Level controllers normally provide control within a predetermined band, or proportional control. On some installations, proportional reset controllers are used for level control.

There are five fundamental types of air-operated controllers: on-off, proportional, proportional with reset, proportional with derivative, and proportional with reset and derivative. This discussion will be limited to the first three types, which are the ones normally used in water works installations.

On-Off Control

Control consists basically of measuring the deviation between the measurement and the control point and then positioning the controlling valve in

order to correct the deviation. The control mechanism provides a means of varying the air pressure to a valve or air motor. Figure 2 illustrates a simple on-off control circuit. It will be seen that there is a flapper and nozzle, the nozzle being fixed and the flapper being linked differentially to the index and pen. When the measurement is above or below the control point, the nozzle is, respectively, uncovered or covered. Air is continuously bled through a restrictor and the flapper nozzle, with a pneumatic relay interposed between the supply and the nozzle. The restrictor imposes a higher head loss than the nozzle, the pressure range at the nozzle being approximately 0–3 psi with a 17-psi supply to the restrictor. When the nozzle is covered by the flapper, a back pressure is exerted on the relay, causing supply pressure to be impressed on

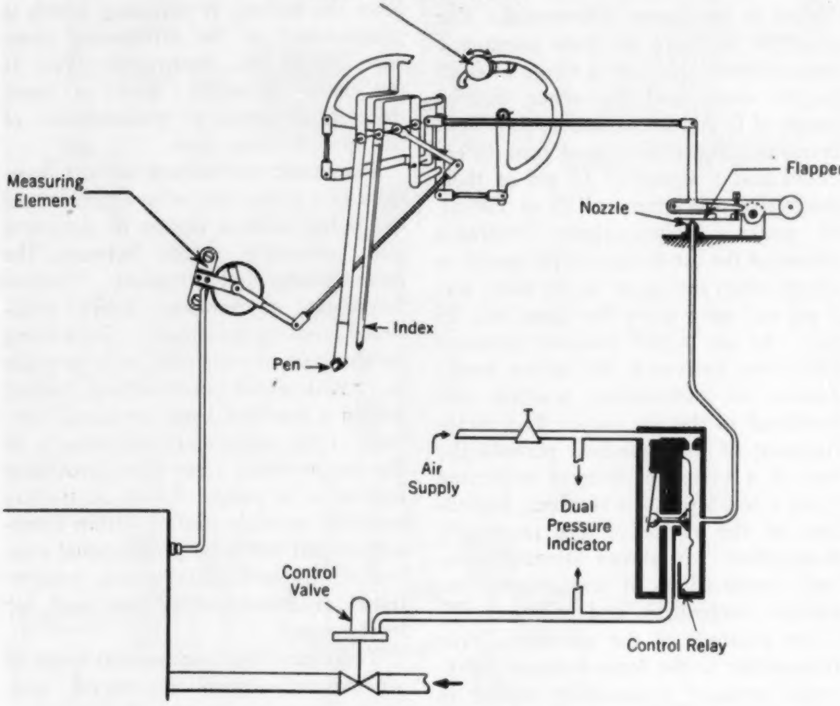

Fig. 2. On-Off Control Circuit

The flapper covers or uncovers the nozzle, depending on whether the measurement is below or above the control point.

the control valve or controlling mechanism. As a result, the control valve opens wide. Likewise, uncovering the nozzle relieves the pressure on the relay and valve motor, causing the valve to close. When the measurement is below the control point, the valve is

wide open; when it is above the control point, the valve is fully closed.

Proportional Control

Because on-off control often proved inadequate, other types of controller have been developed. These cause the valve or controlling mechanism to be so positioned as to keep the measurement within a definite control band of the index. The proportional controller shown in Fig. 3 incorporates all the features of the on-off controller but has a slightly different flapper and nozzle arrangement. In addition, a bellows and opposing spring are included. As the flapper covers the nozzle, the back pressure causes the relay to deliver supply pressure to the valve. Simultaneously, pressure is applied to the proportioning bellows, which moves, causing the flapper to be carried away from the nozzle. This happens in such a short time that the control valve is checked in its movement to a full open position, unless the difference between the measurement and control point is large enough to be outside the range of the proportioning-bellows adjustment. The range over which the proportioning bellows has an effect on the controller output is known as the proportional band and is expressed as a percentage of the chart range. For example, if a pH controller with a chart reading from pH 2 to pH 12 has its

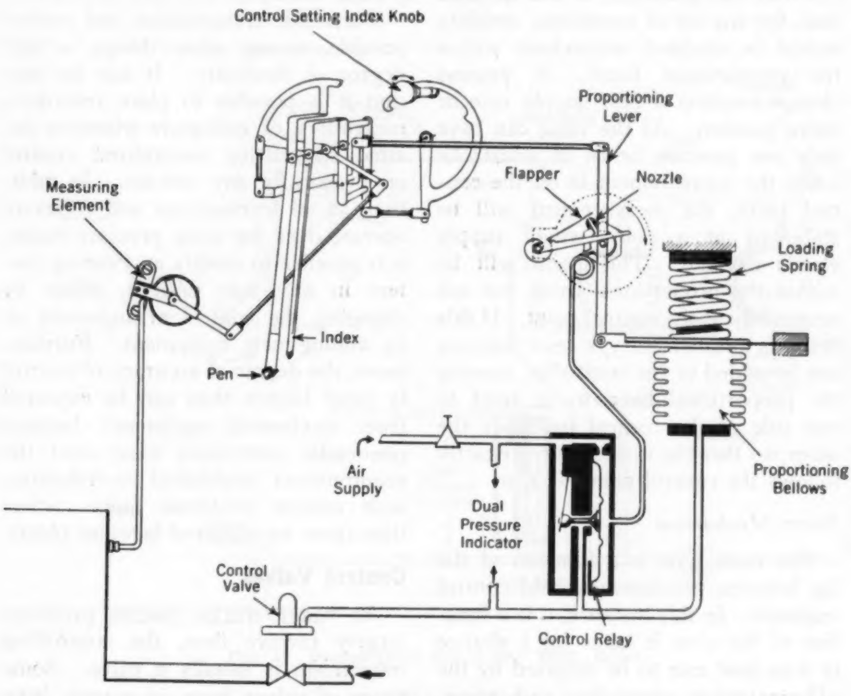

Fig. 3. Proportional Controller

This type of controller keeps the measured variable within a specified range.

control index set at pH 6.5 and the proportional band setting is 50 per cent, the control valve would be just open at pH 4.0 and just closed at pH 9.0. At the control point (6.5), this valve would be at midstroke, while, at any other point between pH 4.0 and 9.0, the valve would be positioned in proportion to the pH measurement.

From this example, it can be seen that, for any set of conditions, stability would be attained somewhere within the proportional band. A process change requires a new supply rate or valve position. As the valve can have only one position or be at midstroke when the measurement is on the control point, the measurement will be stabilized at a point where supply equals demand. This point will lie within the proportional band, but not necessarily at the control point. If this result is unsatisfactory, a reset function can be added to the controller, causing the proportional band to be reset to one side of the control index or the other, so that the measurement exactly follows the control point.

Reset Mechanism

The reset time is a function of the lag between measurement and control response. In this instance, it is a function of the time it takes for a change in lime feed rate to be detected by the pH-measuring electrodes and transmitted back to the control mechanism or valve.

The proportional controller with reset shown in Fig. 4 incorporates all the features previously described, in addition to a reset bellows and adjustable reset resistance. The adjustable resistance or time-delay mechanism compensates for the lag effect. Air leaking through the resistance causes the reset bellows to expand in opposition to the proportioning bellows. This repositions the flapper with relation to the nozzle and, hence, the proportional band with respect to the measurement. The proportional band is offset to one side so that the measurement agrees with the control point index. Proper adjustment of the proportional band and reset results in control at the index setting, regardless of changes in the process variable.

Pneumatic transmission and control provide, among other things, a high degree of flexibility. It can be seen that it is possible to place recorders, controllers, or indicators wherever desired, permitting centralized control on practically any process. In addition, as all transmitters and receivers operate over the same pressure range, it is possible to modify an existing system in any way desired, either by changing the control arrangement or by adding new equipment. Furthermore, the degree of accuracy of control is much higher than can be expected from mechanical equipment, because pneumatic controllers must meet the requirements established by industries with control problems more critical than those encountered in water plants.

Control Valves

As water works control problems largely involve flow, the controlling mechanism is usually a valve. Some types of valves have, of course, been in use in water works for many years, but the advent of automatic control required a reexamination of the control valve problem, with the resultant development of valves suitable for this service.

Figure 5, a semilog plot of valve characteristics—the abscissa is percentage of total flow and the ordinate percentage of total lift, with a constant head loss across the valve being assumed—illustrates some important fac-

tors regarding control valves. It is well known that gate valves are unable to throttle the flow to any appreciable extent when they are more than half open. Because, in control applications, the valve should be effective over the entire range of its position, this type generally does not make a suitable control valve. Where valves with characteristics of this type have to be used on over the entire range of operation, as indicated by the straight-line plot in Fig. 5. Fortunately, the butterfly valve has characteristics very similar to those of the equal-percentage valve and is, therefore, suited for control applications in water treatment plants.

This uniformity characteristic of a valve is not so important from the standpoint of adjusting a controller,

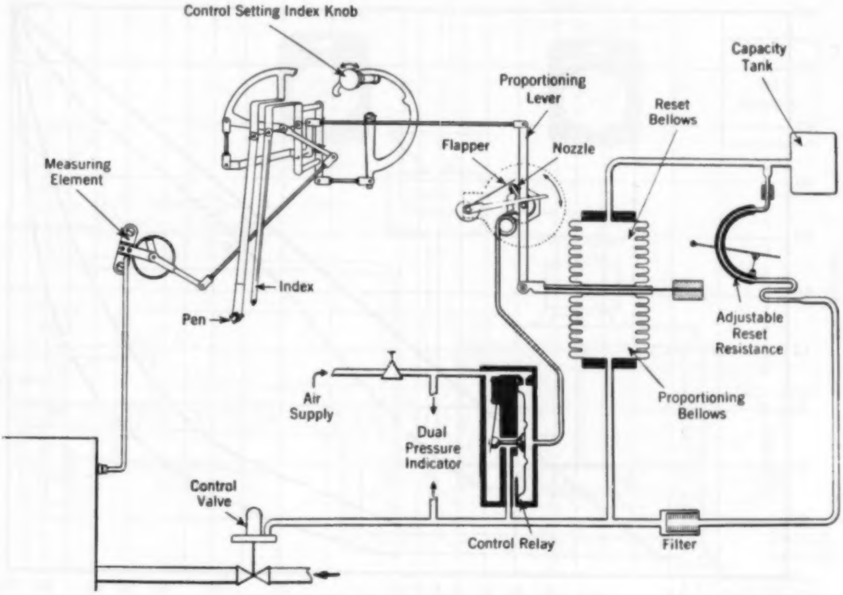

Fig. 4. Proportional Controller With Reset

The reset mechanism causes the proportional band to be shifted with respect to the control point.

fluids containing solids, it is necessary to readjust the controller setting for changes in control point or serious changes in loading conditions. In addition, it may be necessary to select a valve with an odd size in order to produce satisfactory flow control. These two problems led to the development of the equal-percentage characterized-port valve, which behaves uniformly as hydraulic conditions within most filter plants do not change sufficiently to require controller settings to be changed. It is important from the viewpoint of valve selection, however. The actual hydraulic conditions under which a valve will operate are rarely known, and it is customary to use line-size valves in treatment plants. Consequently, a valve with equal-percentage

characteristics can be used with assurance that it will operate efficiently. The use of the gate valve for control purposes should not be seriously considered.

The "rangeability" of a valve is generally considered to be the upper limit of the straight section of the curve on a semilog plot, divided by the lower limit. Butterfly valves have rangeabilities on the order of 20:1, whereas gate valve rangeability is on the order of 5:1. In other words, the characteristics of a butterfly valve, with regard to lift versus flow, are uniform over approximately four times the range of gate valves. The butterfly valve has a small number of wearing parts, the shafts being equipped with bearings and lubricators. These mechanical features, coupled with favorable control characteristics, make it the most satisfactory type of valve for use in water works control applications.

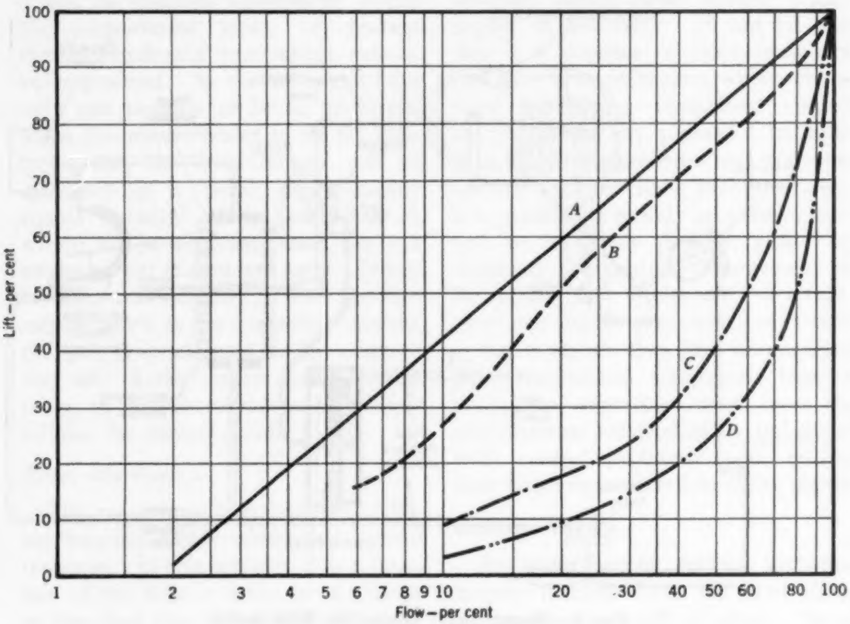

Fig. 5. Valve Characteristics

Key: A—*equal-percentage characterized-port valve;* B—*typical butterfly valve;* C—*typical gate valve;* D—*typical "Saunders" patent valve.*

Control Applications

Taking as an example a filter plant utilizing chemical coagulation, sedimentation, and rapid sand filtration control, the first treatment process is the control of chemical coagulation. As is well known, coagulation takes place at an optimum pH value. The diagram in Fig. 6 illustrates a typical pH control system, with the measurement being made beyond the flash mixers. The controller operates the lime feeder in such a manner as to provide

a final pH of optimum value for coagulation; a valve controls the addition of a solution or adjusts the speed of operation of the chemical feeder through the use of a d-c motor operating from a rectified a-c source. With a pneumatically actuated rheostat in the circuit, the speed can be controlled from the output of a pneumatic controller. The feeder can be equipped with a pneumatic operator for changing the feed rate. Several well known types of feeders are available with such pneumatic operators. If the character of the water changes over a period of time and it is found that there is another optimum pH control point, it is necessary merely to adjust the index setting of the pH controller.

The dosage of coagulant is generally in proportion to the rate of flow—that is, so many grains per gallon or parts per million. In Fig. 6, proportional feeding is obtained through the use of a differential-pressure pneumatic transmitter. This instrument transmits the flow rate reading to a timed-pulse controller, which operates the alum feeder for a fraction of time directly proportional to the flow rate. At 100 per cent of flow, a contact is closed continuously; at 50 per cent, the contact is closed 50 per cent of the time (30 sec, if the controller has a 1-min cycle). If

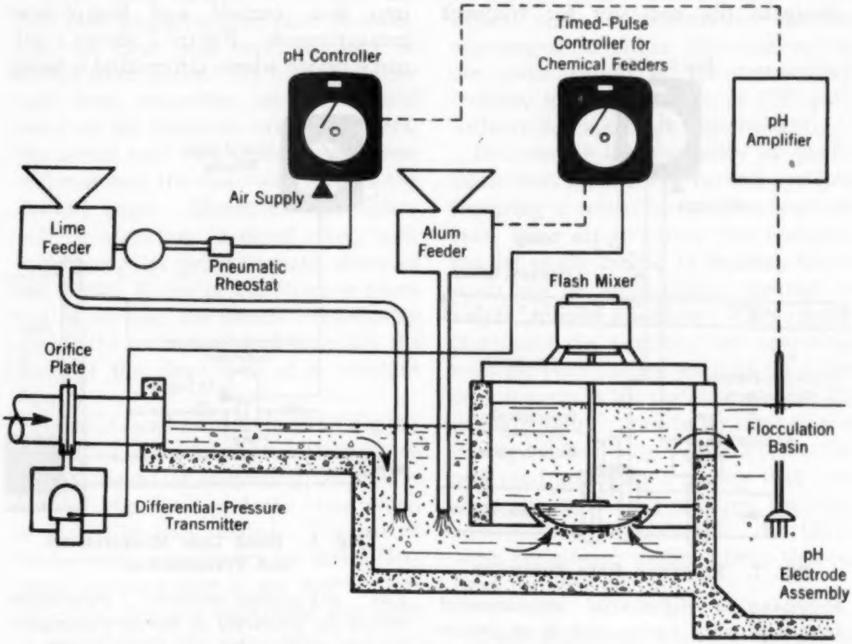

Fig. 6. Coagulation Control

The differential-pressure pneumatic transmitter sends the flow rate reading to the timed-pulse controller, which operates the alum feeder for a fraction of time directly proportional to the flow rate.

a feeder motor is operated through this contact, it feeds in proportion to flow.

If a pneumatic controller were used, the alum feeder would be equipped either with a pneumatic rheostat and d-c motor or with a pneumatic operator for changing the feed rate. The control of those two variables in the chemical coagulation process would eliminate the necessity for frequent checking of the dosage rate of coagulant and the pH of the water in the coagulation basin.

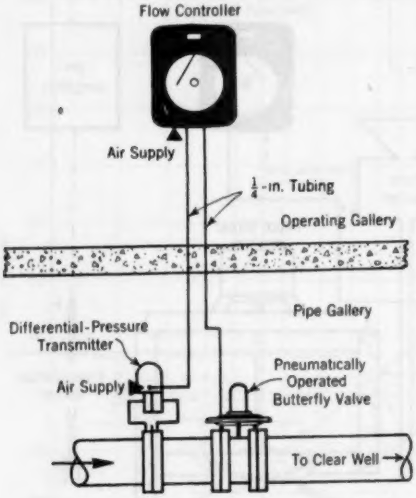

Fig. 7. Filtration Rate Controller

Receiving a differential measurement from a primary device such as an orifice plate or venturi tube, the flow controller transmits a signal to a valve located downstream.

In general, the hydraulic gradient throughout a filter plant is determined by the level in the primary treatment basin. For this reason, it is necessary to control the level of the sedimentation basins at a point where sufficient hydraulic gradient is provided. This is easily done by a pneumatic controller—with or without reset—operating a control valve on the inlet to the sedimentation basin. The controller may be mounted locally or on the central control board and can be equipped as either a recorder or an indicator.

The greater portion of the control in a treatment plant is required on rapid sand filters. This resolves itself into flow control and loss-of-head measurements. Figure 7 shows a primary device whose differential is being transmitted to a flow controller. This instrument, in turn, is transmitting a signal to a control valve located downstream from the primary device, in order to maintain a constant flow rate through it. This arrangement is an example of the basic type of flow control circuit. The primary device can be an orifice plate, flow nozzle, or venturi tube, the selection being based largely upon head loss considerations and the character of the fluid. It will

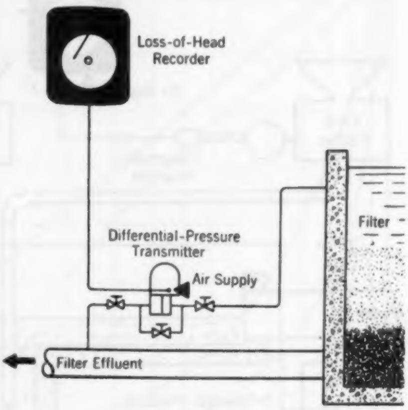

Fig. 8. Head Loss Measurement and Transmission

The differential-pressure transmitter avoids the problems of mercury manometers and mechanical transmission.

be seen that the controller can be located at any desired point in the plant, permitting centralized control of the flow rate to the filter. There are no cables, pulleys, or the like; the only connection between the transmitter and the controller, and between the controller and the valve, is through ¼-in. tubing.

It is often desired to control the flow rate through a filter plant to maintain the clear well level constant or within fixed limits. By having a clear well level controller set the control point of the filtration rate controllers, the actual total rate through the plant will maintain the clear well level at the desired point. Moreover, the filters will be operating at equal rates, with each doing its proportionate share of the work. If one of the filters is taken out of service, the others immediately divide the extra load and maintain the level of the clear well at a constant figure.

Loss-of-head measurements (Fig. 8) are extremely important, for they provide a means of indicating the condition of the filter and the time when it should be backwashed. The use of the mercury manometer for this differential measurement is not entirely satisfactory, because each time the filter is backwashed the manometer is overranged in the reverse direction. It is necessary, therefore, for the check valves on the manometer to remain tight during this operation. Although the check valves can be relied upon to do this, grains of sand, which are quite often present, may score the seats, causing leakage; and leakage can result in the loss of mercury. Another consideration is the transfer of the float movement from the mercury manometer to a gage. In mechanical systems, this is normally done with cables, which are subject to stretching. The above remarks on check valves do not hold for float wells and float-and-cable instruments, but the maintenance of cables and pulleys is necessarily a serious problem.

The use of a nonmercury differential-pressure transmitter eliminates these problems. It does not rely upon check valves, and the transmitter can be overranged in either direction up to the static rating of the transmitter (which, in this instance, is 750 psi) without damage to the instrument.

Because of the flexibility of pneumatic instrumentation, control systems requiring a minimum of attention—or even none at all—from the operator can be easily built. If desired, backwash can be accomplished through a single selector switch. This setup eliminates the necessity for operating multiple pilot valves, while at the same time it permits the operator to give his full attention to preventing filter upsets during backwashing. Where the water and sand are of such quality that constant attention is not required, the filter controls can be automatic, the backwash cycle being initiated from loss-of-head measurements.

This discussion has not attempted to review all of the possible applications or methods of use of pneumatic transmission and control in water treatment plants. There are numerous others, of varying interest and importance. Because of its flexibility, this standard tool of industry is becoming one of the more important means by which the efficiency of the water treatment process may be increased, to the benefit of both the consumer and the utility.

Flow Tests on Distribution Systems

By W. D. Hudson

A paper presented on Mar. 24, 1953, at the Southeastern Section Meeting, Macon, Ga., by W. D. Hudson, Asst. Chief Engr., The Pitometer Associates, Inc., New York.

IN a complete analysis of a distribution system, several types of flow tests should be made, including: tests of pumping equipment efficiency; measurements of flow in the various trunk mains; fire flow tests, to determine the amount of water available for fire-fighting purposes; loss-of-head tests, to determine the roughness coefficient of the pipe; pressure recordings, to determine the pressure variations at critical points; simultaneous pressure and flow measurements, to determine the hydraulic gradient in various sections of the system; and tests of master meter accuracy.

Pumping Efficiency

Most water pumping plants employ the centrifugal type of pump, either electrically driven or powered by a diesel motor. In some of the larger cities, steam turbine drives are used. A centrifugal pump is designed to work under rather rigidly fixed conditions, any appreciable variation from which is accompanied by a decrease in efficiency. The subject of efficiency is too frequently overlooked by plant operators as long as the pumps do their work.

It might be well, at the outset, to review the basic principles of a centrifugal pump. The water entering the pump through the suction inlet is seized by the impeller vanes and spun until it is rotating at terrific velocity. As pressure, not speed, is desired, the pump is so designed that, by gradually slowing down the water velocity in the casing of the pump, the energy stored up in the rapidly moving water is almost entirely changed to pressure. The pressure imparted to the water depends on the diameter of the impeller and the number of revolutions per minute that it makes.

There are, of course, practical limits to both speed and diameter, and, if the desired pressure cannot be obtained in one pump, two are used in series. If they are in one casing, each pump is called a stage. As electrically driven centrifugal pumps run at a constant speed, there is always one point at which they are most efficient.

Figure 1 shows a typical centrifugal-pump curve. It will be noted that the greatest efficiency for this pump—approximately 87 per cent—is secured when the head is approximately 90 ft and the discharge rate is 4,200 gpm; the efficiency is at least 85 per cent as the head varies between 95 and 78 ft and the discharge rate between 3,600 and 5,100 gpm. It will also be noted that, if the head drops to 60 ft, the discharge will increase to 6,000 gpm, but the efficiency will be only 75 per cent; if the head is increased to 105 ft,

the pump will put out 1,750 gpm and will be only 58 per cent efficient. If the head is raised an additional 2 psi (to 109 ft) the pump will not discharge any water.

One can readily see that each centrifugal pump must be designed for the particular job it is to do, and that usually it is false economy to employ a pump that does not have the right characteristics for the job. Often an efficiency test on a pump will bring to light conditions that can be corrected with a minimum of expense, thereby greatly improving the operation of the entire distribution system.

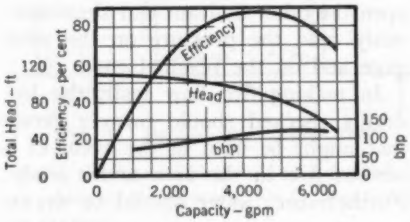

Fig. 1. Pump Performance

The curves shown are typical for centrifugal pumps.

An approximate check on the efficiency of a pump can be made by dividing the monthly pumpage, in gallons, by the monthly power consumed, in kilowatt-hours. Theoretically, 1 kwhr will raise 318,200 gal of water 1 ft if the motor and pump are 100 per cent efficient; dividing 318,200 by the total head, in feet, with the pump running will give the theoretical gallons pumped per kilowatt-hour. When the actual gallons pumped per kilowatt-hour is divided by the theoretical value, the result is the wire-to-water efficiency of the pump and motor.

A more accurate method of testing efficiency is the following:

1. Measure the output of the pump during the test period with the master meter.

2. Obtain the total head by: (a) measuring the head on the outlet side of the pump with an accurate pressure gage and multiplying by 2.31 to convert pounds per square inch into feet of water; (b) measuring the suction lift of the pump in inches of mercury and converting to feet of water by multiplying by 1.13; and (c) measuring the difference in elevation (in feet) between the gage on the outlet and the connection of the mercury column on the inlet. The sum of the (a), (b), and (c) values is the total head.

3. Divide 318,200 by the total head, to get the theoretical gallons pumped per kilowatt-hour.

4. Determine the number of kilowatt-hours of power used during the test. (The power company will usually determine this factor or advise how it can be obtained from the reading of the electric meter.)

5. Divide Value 1 by Value 4, to find the gallons actually pumped per kilowatt-hour.

6. Divide Value 5 by Value 3, to obtain the wire-to-water efficiency.

Trunk Main Flow

The measurement of the flow of water through the trunk main system is of the utmost importance in the study or analysis of a distribution system. The only feasible way of making this type of test is by using pitometers. The instrument is inserted in the main through a 1-in. corporation cock. It is a reversible pitot tube and can therefore measure reverse flows in the main. A recorder may be used to provide a continuous chart of the flow at the gaging point for any period of time desired—usually 24 hr. Thus, the

maximum and minimum rate and the total flow for the period can be obtained at the gaging point. From simultaneous readings at a number of points, it is determined exactly how the water circulates through the trunk main system.

These flow measurements show which mains are doing their share of the work, which are overloaded, and which are not carrying the amount of water that would normally be expected. The amount of water withdrawn from the trunk main between gaging points can be found by comparing the simultaneous measurements. Such tests often reveal closed valves, obstructions, air pockets, or other easily remediable conditions that may be seriously affecting the system. In addition, the measurements furnish the flow data necessary for the accurate design of reinforcements and extensions to the system. Special equipment is required, as well as engineers trained to conduct the tests and interpret the results.

Fire Flow

Fire flow tests are the one type of distribution flow test that is easy to make and does not call for a great deal of equipment. Four dial pressure gages and several hydrant pitot tubes are all that is needed for most tests. Because of the relative simplicity of the procedure and computations, these tests are invaluable to the average water works man in determining whether or not the water distribution system is adequate.

From a fire flow test, the pressure and flow of water available for fire protection at various points in the system can be learned, as well as the location of deficiencies. In most cities, the National Board of Fire Underwriters or its affiliates have made such tests. If they are rerun, the results of the two series can be compared to find out whether the system has deteriorated or become overloaded at some points. Frequently, closed valves prove to be the trouble.

The fire flow test procedure may be briefly described. At the point where the amount of water available for fire protection is to be determined, place a gage on a fire hydrant. Then place a hydrant pitot gage in the nozzle of another hydrant. A pressure gage can be used instead, but it must be in the same plane as the nozzle through which the water is drawn. Observe the pressure on the residual hydrant; then open the other hydrant and simultaneously read the pressure on the pitot gage and on the residual gage.

In making fire flow tests, the hydrants checked should form a group that might be used in the event of a serious fire in the area under study. Furthermore, water should be drawn at such a rate as to create a drop in pressure large enough so that it will not be measurably affected by normal fluctuations in the system. It is probable that several hydrants will have to flow simultaneously to provide a sufficient pressure drop for an accurate test.

From the pitot gage pressure reading, P (psi), and the diameter of the nozzle, D (in.), the hydrant discharge, Q (gpm), can be found in an appropriate table or computed from the formula: $Q = 27D^2\sqrt{P}$.

To find the approximate amount of water available at 20-psi residual pressure (the minimum that a system should carry) the following formula can be applied:

$$Q_r = Q_t\left(\frac{P_c - 20}{P_c - P_o}\right)^{0.5}$$

in which Q_r (gpm) is the discharge at 20-psi residual pressure, Q_t (gpm) is the total discharge of the open hydrants, P_c (psi) is the pressure at the residual hydrant with the other hydrants closed, and P_o (psi) is the residual-hydrant pressure with the other hydrants open.

As most of the loss is in the main system rather than in the hydrant outlets, Q_r can be calculated more accurately from the relevant Hazen-Williams formula, which is the same as

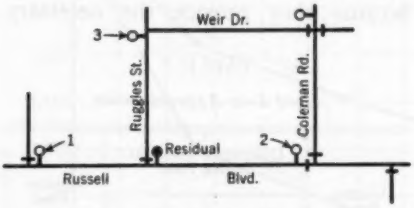

Fig. 2. Typical Fire Flow Test

Using the data in Table 1 and substituting in either of the formulas for Q_r *mentioned in the text:*

$$Q_r = 1{,}740\left(\frac{72-20}{72-48}\right)^{0.5}$$

$$= 2{,}540 \text{ gpm}$$

or:

$$Q_r = 1{,}740\left(\frac{72-20}{72-48}\right)^{0.54}$$

$$= 2{,}640 \text{ gpm.}$$

the preceding equation except that, in place of an exponent of 0.5 (the square root), an exponent of 0.54 is used. Figure 2 illustrates a typical fire flow test situation, with the data shown in Table 1.

Head Loss

Loss-of-head tests are made to determine the internal condition of the pipes—that is, to learn whether they are tuberculated or obstructed. A thoroughly trained engineer is required for this type of work.

In making a loss-of-head test, a pitometer, to which are attached a manometer and a very accurate recording pressure gage, is inserted in each end of the section of main to be tested (Fig. 3). Then the gate valves on the distribution mains are closed, so that no water will be taken off between the test points. For a given period of time the deflection on the manometer and the head on the pressure gage are read simultaneously at each end of the test section. At the completion of this test,

TABLE 1

Fire Flow Test Data

Hydrant*	Observed Pressure—*psi*		Calculated Discharge† *gpm*
	Residual Hydrant	Discharge Hydrants	
1–3 closed	72		
1		14	630
2		12	580
3		10	530
1–3 open	48		1,740

* All nozzle diameters are 2½ in.

† For Hydrant 1, discharge equals $27 \cdot (2\tfrac{1}{2})^2 \cdot \sqrt{14} =$ 630 gpm. Values for the other hydrants are calculated in the same way.

the gages are checked. The gages are then reversed and another, independent test is made. It is necessary to know the exact elevation at each gaging point. To the elevation at each end is added the pressure reading, giving the water level at each of the test points. The difference in water level between the two points is the loss of head in that section of main. When the loss of head is divided by the length of pipe tested, the hydraulic slope of the pipeline is obtained. This result is expressed in feet of head lost per 1,000 ft of pipe. The flow velocity is found

from the pitometer reading. If the slope, velocity, and pipe diameter are known, the Hazen-Williams roughness coefficient, C, can be obtained from tables or calculations.

Another method of conducting loss-of-head tests, for short pipe sections (500–1,000 ft), involves laying a parallel pipe, usually $\frac{1}{2}$- or $\frac{3}{4}$-in. in diameter. The pitometer is inserted in one end of the section of main to be tested, and one end of the parallel pipe is also connected at this point. Somewhere in the parallel pipe it is necessary to install a glass U tube containing mercury or some other liquid heavier than water. The other end of the parallel pipe is connected to the main at the end of the test section. The loss of head is obtained by reading the deflection (in inches) of the liquid in the U tube. This deflection must then be converted into feet of water. The velocity is obtained from the pitometer reading. As the size of the main being tested and the length of the parallel pipe are known, Hazen-Williams C can be obtained.

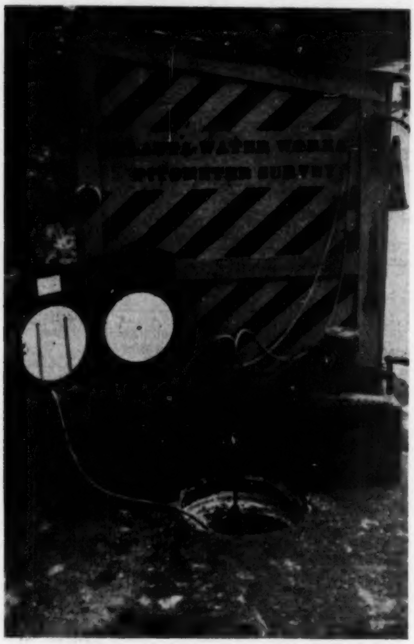

Fig. 3. Head Loss Test

The pitometer, with manometer and recording pressure gage attached, is set up at each end of the pipe section being tested.

Pitometer loss-of-head tests are extremely important in system design because they provide the necessary basic data. No one can guess the internal condition of a pipe, and visual inspection is of little value in estimating its coefficient. Although the Hazen-Williams tables of coefficients based on age are valid for the type of water specified, the quality of water varies so much from place to place that the only certain way to determine the coefficient of a pipe is to make a loss-of-head test.

TABLE 2

Head Loss Approximation

Gaging Point	Observed Pressure at Gaging Point		Approx. Loss *psi*
	Max. (Night) *psi*	Min. (Day) *psi*	
1	72.5	41.5	31.0*
2	73.5	32.5	41.0*
		Difference	10.0†

* Pumping station to gaging point.
† Between gaging points.

Such tests will also indicate whether a cleaning program to restore the approximate original pipeline coefficient would be of benefit. Quite often a main-cleaning program will eliminate

or delay the need for adding to the system.

Pressure Recording

The recording of pressure at critical points in a water distribution system should be considered a necessity. In this way, a constant check can be kept tem is becoming overloaded or that the trunk mains are losing their carrying capacity. Permanent 7-day recording pressure gages should be installed in all pump stations and usually in each fire station in the city. The fire stations, as a rule, are fairly evenly spaced throughout the city and are good locations for such gages.

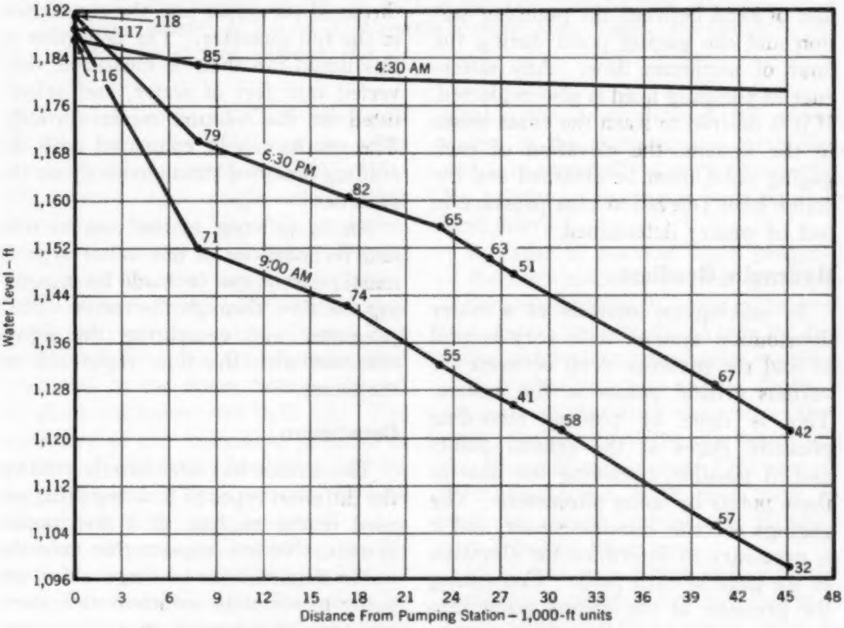

Fig. 4. Hydraulic Gradients of Distribution System Section

The numbers at various points on the curves indicate observed pressure, in pounds per square inch. To find the head loss (in pounds per square inch) between the pumping station and a given point, divide the difference in water level by 2.31.

on the system. By comparing the pressure gagings from time to time, it can readily be seen whether the pressure pattern is changing. A sudden and prolonged decrease in pressure is a good indication that a sizable leak has developed. If the pressure drops gradually, it is probable that the sys-

If there are complaints in a particular part of the system, it is a good policy to run 24-hr pressure gagings, usually at two points simultaneously—one in the area in which the trouble exists, and the other at the connection of the feeder main supplying the area. By comparing the charts, the loss between

the points can be determined and may give a clue to the trouble.

If the pressure at the pumping station is nearly constant, the approximate head losses between the pumping station and the gaging points, and between the gaging points themselves, can be found quickly by the method illustrated in Table 2. These approximations do not take into account the loss of head between the pumping station and the gaging point during the hour of minimum flow. Any difference in pumping head is also neglected. If it is desired to learn the exact losses in the system, the elevation of each gaging point must be obtained and the water level (elevation plus pressure in feet of water) determined.

Hydraulic Gradient

In making an analysis of a water distribution system, it is very helpful to find the pressure drop between the various critical points in the system. This is done by placing recording pressure gages at the critical points and, if possible, obtaining the flow at these points by using pitometers. The gagings are run simultaneously, and it is necessary to determine the elevation of the gage at each point. Converting the pressure at the gaging point into feet of water and adding this figure to the elevation, the water level can be obtained. These values can then be plotted on suitable graph paper, and the results will be the hydraulic gradient.

Figure 4 shows the hydraulic gradients for part of a water system at different times. It will be noted that there is a large loss of head between the pumping station and the first gaging point. The logical step to take in reinforcing this system is to provide additional main capacity leading from the pumping station.

Master Meter Test

It is very important that the master meters be checked periodically for accuracy. If the meter is a venturi type, the recorder can be tested by connecting a U tube, partially filled with a liquid heavier than water, between the throat of the meter and the connection to the full diameter. The deflection of this liquid can then be measured, converted into feet of water, and substituted in the venturi meter formula. The results can be compared with the reading obtained simultaneously on the recorder.

An overall test of the venturi tube and recorder, or of any other type of master meter, can be made by measuring the flow through the meter with a pitometer and comparing the figure obtained with the flow registered on the meter.

Conclusion

This article has very briefly covered the different types of flow tests that are used in the analysis of a distribution system. No one expects that even the water departments in large cities are going to maintain sufficient equipment and trained personnel to make a complete study of their distribution systems. It is simply not economically feasible to do so. It is much more economical to consult an engineering firm occasionally and obtain the benefits of the experience and technological knowledge of others who are constantly working on water distribution problems. In view of the present cost of materials and high taxes, it is more than ever essential that proposed improvements be thoroughly engineered.

Diatomaceous-Earth Filtration in New York State

By J. Kenneth Fraser

A paper presented on Sep. 10, 1953, at the New York Section Meeting, Lake Placid, N.Y., by J. Kenneth Fraser, Civ. Engr., Barker & Wheeler, Engrs., Albany, N.Y.

IN certain areas of California, millions of years ago, and in other locations more recently—possibly 100,000 years ago—single-celled algae known as diatoms grew in profusion and in a variety of shapes. During their brief life, they acquired a "skeleton" formed from silica which they extracted from the water. Over a long period of time, as the diatoms died and sank to the ocean floor, a deposit of great thickness was built up. The structure of the individual particles is patterned after the formation of the original plant. This material is now mined, crushed, and processed for a number of purposes, including the filtration of water and other liquids.

A diatomaceous-earth filter may be contrasted with a sand filter, in which the sand bed is supported upon layers of graded gravel. In a diatomaceous-earth filter, the gravel is replaced by an element with minute openings—wire-wound plastic or steel cores, screen mesh cylinders, sintered brass or sintered stone (carborundum), or, in some instances, cloth. Instead of the sand, a film of diatomaceous earth is formed on the filter elements by applying a slurry or mixture of diatomaceous earth and water to them. Filtration is then accomplished by passing the water through the minute spaces in the diatomaceous-earth film, which is essentially a very fine screen. No gelatinous coating is built up as in sand filters.

In order to prevent rapid plugging of the filter medium, smaller quantities of diatomaceous earth, again in the form of a slurry, are introduced into the raw water as filtration takes place, resulting in a gradual increase in the depth of the film on the filter element. Obviously, the nature of the raw water and the rate of filtration control the amount of diatomaceous earth that must be used to obtain satisfactory filtration and extended filter runs.

Most diatomite filters utilize a steel shell that is divided into two compartments separated by a metal plate or diaphragm which supports the cylindrical elements. Such a design allows as much as 150 sq ft of filtering surface to be built into a unit with a diameter of approximately 2½ ft, thus requiring only a relatively small amount of floor space.

The diaphragm that supports the filter elements and also provides a mechanical separation between the raw and the filtered water is one point of structural weakness in a diatomite filter, as it must withstand the head loss

due to filtration. In some designs, this diaphragm is subject to failure if head losses exceed 30 psi. Therefore, when the loss approaches this figure, the filter must be cleaned. This is done by reversing the flow, thus removing the earth and dirt from the element, the used diatomaceous earth being discharged to waste. Backwashing can be accomplished in a very few minutes, and the filter can then be recharged and put back in operation. Relatively small quantities of water are required for backwashing, as compared with rapid sand filters.

Tupper Lake Installation

To illustrate the conditions under which diatomite filters might be selected in place of the more conventional methods of filtration, a typical application, at Tupper Lake, N.Y., will be described.

Tupper Lake, a village with a population of 5,500 is situated in the Adirondack Mountains in northeastern New York, approximately 31 miles southwest of Lake Placid. It is a commercial and resort center, with industries engaged in the manufacture of wood products, and is also the site of the US Veterans Administration Sunmount Tuberculosis Hospital, with a capacity of 500 patients.

Tupper Lake has two sources of gravity supply, which, being incapable of providing sufficient water to meet peak seasonal requirements, were formerly supplemented by an emergency pumping station on the Raquette River. During the summer Raquette River water was tepid and undesirable for drinking and, in addition, was subject to pollution from recreational use, so that it was bacteriologically unsafe except when heavily dosed with chlorine.

The possibilities of developing additional sources and expanding the existing gravity supplies were investigated. It was ultimately decided that additional water could best be obtained from Tupper Lake, which is approximately 7 miles long and 2 miles wide and is located approximately 2 miles southwest of the village. Although permitting utilization of water from Tupper Lake, the New York State Water Power and Control Commission required that the supply be filtered and disinfected with chlorine.

Three methods of filtration were considered: slow sand, rapid sand, and diatomaceous earth. The initial cost of both types of sand filtration was higher than that of diatomaceous earth, and this fact, coupled with the necessity for providing a plant that could be readily started and stopped to act as an auxiliary supply, led to serious consideration of diatomaceous earth filters. Moreover, Tupper Lake water was of good quality, practically unpolluted and generally low in turbidity, although somewhat high in color. For such water, diatomaceous earth filtration was particularly well suited. It was, therefore, decided to proceed with the construction of an intake, pumping station, and filtration plant on the shore of Tupper Lake.

A 14-in. standard mechanical-joint, cement-mortar lined, cast-iron intake line was installed on the bottom of the lake, extending approximately 800 ft offshore and taking water from a depth of approximately 33 ft, or 10 ft above the lake bottom. The pumping station and filter building, approximately 29 by 45 ft, included a substructure that consisted of a three-compartment

wet well and screen chamber. Water flows into the wet well by gravity through the 14-in. intake and is picked up by three close-coupled, deep well turbine pumps, one of 0.5-mgd and two of 0.75-mgd capacity. This type of pump not only did away with priming difficulties common to a conventional centrifugal-pump installation but could also operate with good efficiency while a 25-psi pressure loss was gradually building up through the diatomaceous-earth filters. Furthermore, the installation was capable of being altered conveniently to meet reduced heads resulting from future distribution system improvements.

Four diatomaceous earth filters were installed, each with a filtering area of 153 sq ft, or a total of 612 sq ft in all. A filtration rate of 2 gpm per square foot would provide a capacity of 1,224 gpm, or 1.75 mgd.

In addition to the pump and filter room, the building included separate rooms for the heating system, lavatory, and chlorination facilities. A gas chlorinator furnishes proportional chlorination at the inlet to the wet well. The entire cost of land, intake, building, and pumping and filter equipment for this 1.75-mgd plant amounted to less than $125,000, which was substantially below the estimated costs for either a rapid or a slow sand filter plant.

Operation of Filters

Although a diatomaceous-earth filter plant has certain limitations, it affords a practical and economical means of treating Tupper Lake water. The system is flexible in operation, with variable filtration rates. Such a plant can be put into full operation within a matter of minutes and will run at full capacity immediately. The Tupper Lake installation is operated only when the present gravity supplies are inadequate or of unsatisfactory quality.

When operated at a rate of 0.5 mgd (0.57 gpm per square foot of filter surface) the filters have a maximum run of 87 hr before backwashing is needed. It must be realized that the length of filter runs between backwashings is, of course, dependent upon the rate of filtration, character of the water, time of year, and condition of the lake in general.

A minor reduction in color has been noted in the filtered water at the Tupper Lake plant. A diatomaceous-earth filter cannot, however, be depended upon for color removal and should not be used if color in the raw water is above acceptable limits (20 ppm).

The operating procedure followed at Tupper Lake consists of charging each filter, before filtration, with a precoat of diatomaceous earth, amounting to 2 oz per square foot of area (19 lb per filter), to cover the carborundum tube elements used in this installation. After the precoat has been added, one or more of the pumping units is started, and the water is allowed to pass gradually through the filters and run to waste. This is done for approximately 2 min. The waste water is run back into the lake through a 10-in. cast-iron drain extending approximately 200 ft offshore. This drain line is also used to carry the backwash from the filters out into the lake for disposal.

The film of diatomaceous earth is held on the filter elements by the pressure of the water as it passes from the lower portion of the filter, through the tubes and the baffle plate, to the upper portion. After each filter has been precoated and placed in operation, a

slurry feed, composed of diatomaceous earth in suspension, is introduced into the raw-water header. The density of this "body feed" depends entirely upon the condition of the raw water and is not necessarily proportional to the amount of water filtered.

Under average conditions, 300–400 lb of filter aid is used for each million gallons of water treated.

To protect the vulnerable metal diaphragm, the maximum pressure differential permitted between the inlet and outlet side of the filters is 30 psi. Because the Tupper Lake station was designed for operation without constant attendance, it was necessary to provide some means of shutting down the pumps when the pressure differential through the filters approaches 30 psi. A mercury switch with a variable setting stops the pumping units automatically when the difference in pressure between the raw and filtered water reaches a predetermined level.

The slightest reversal of flow through the elements results in the loss of the diatomaceous-earth coating. Such a condition would normally occur when the pumps are stopped, thereby making it necessary to replace the filter aid before filtration could again be started. To overcome this problem, a 30-gpm circulating pump, taking suction from the discharge side of the filters and discharging into the influent side, was installed. This pump runs continuously while the filters are in operation and prevents the loss of the filter aid during the interruptions in the operating cycle. This method of holding the filter cake on the elements has proved entirely satisfactory.

Backwashing is accomplished by utilizing water from the distribution system. Quick-opening valves on each filter inlet provide an explosive backwash, which reverses the operation of the filter and blows the diatomaceous earth out of the element openings into a waste water pipeline. After a short flushing period the filter is ready to be recharged and placed in operation again, without any appreciable loss of time in the operating cycle.

Willsboro Installation

A small diatomaceous-earth filter plant has been installed at Willsboro, N.Y., a community of approximately 900 people, located on the western side of Lake Champlain approximately 1½ miles south of Willsboro Bay. This community has been interested in establishing a water supply system for more than 20 years.

After considerable preliminary investigation, it was determined that the best source of water supply appeared to be Willsboro Bay on Lake Champlain. Again, filtration was needed to meet the requirements of the governmental agencies having jurisdiction, and, with the limited amount of money available and the good quality of the lake water, diatomaceous-earth filtration offered the best solution.

An 8-in. cast-iron intake was installed at the bay and a small pumping station and filter building were constructed on the south shore. The station consisted of two pumping units of the same type as at Tupper Lake, one having a capacity of 100 gpm and the other 200 gpm. A single filter unit with 115 sq ft of filtering area was installed. It is contemplated that a second unit of the same size will be added in the future.

In this location, the water, after filtration, is pumped through approximately 8,000 ft of 6-in. asbestos-cement transmission line to the distribution system. The distribution facilities

comprise approximately 4 miles of 6-, 8-, and 10-in. mains and a 200,000-gal steel storage tank. The total cost was approximately $185,000; it is evident that such a system could not have been built with the money available had it been necessary to utilize rapid or slow sand filtration.

The operation of the 100-gpm pump is sufficient to meet normal requirements. This means that the single filter unit is operated at a rate of slightly less than 1 gpm per square foot. The average length of filter run is approximately 17 hr, although there have been maximum runs as long as 68 hr between backwashings. The unit at Willsboro has been in continuous operation since November 1952, and, except for minor operating difficulties, has proved entirely satisfactory.

Economic Considerations

High construction prices, as well as legal and other restrictions on the amount of money that can be spent by small communities for water supply purposes, frequently dictate very rigid economies in the scope and design of a project. With filtration usually being required for surface water subject to pollution, the incorporation of diatomaceous-earth filters in the plans for such a supply, particularly when the character of the raw water lends itself to this type of filtration, warrants careful study.

Most of the diatomaceous earth used in filters is mined and processed in California. This fact, coupled with the high cost of transportation, makes diatomaceous earth rather expensive in the East. It sells there for approximately 6–7 cents a pound and was in short supply during 1953.

In the general vicinity of Willsboro, there occurs a natural form of calcium silicate, called Wollastonite. The only known deposit of commercial importance, its size is conservatively estimated at 15,000,000 tons. This fibrous material has been employed in the paint and ceramics industries, and is being considered for use in making silica gel. Experiments with Wollastonite as a substitute for diatomaceous earth have recently been undertaken at Willsboro. If satisfactory results are obtained, the cost of operating this type of filter will be reduced considerably.

Conclusion

There are four diatomaceous-earth filter installations in use on public water supplies in New York State—at Willsboro, Tupper Lake, Cherry Valley, and Gasport. The 1.75-mgd plant at Tupper Lake is by far the largest of these and, in fact, is one of the largest municipal plants of this type in the country.

From the author's experience, which is limited to the installation and operation of the Tupper Lake and Willsboro plants, it would seem that, where intermittent filtration must be provided and where the water is of a character that lends itself to filtration by diatomaceous earth, such a process merits serious consideration.

Acknowledgment

The author would like to express his appreciation to the officials at Tupper Lake: G. Ward Yeomans, formerly Municipal Manager; Oliver Prespare, Village Clerk; Gordon Bisson, Village Supt.; and William Cassell, Plant Operator; to Livingston Hatch, Supervisor, Town of Willsboro; and to others who have helped to furnish the data necessary for the preparation of this paper.

Discussion

C. R. Cox and C. S. Maneri

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This discussion is based primarily on experience with the diatomaceous-earth filters at Cherry Valley, N.Y., which have been operated since 1949 and which represent the first installation in the United States for the treatment of a public water supply. Experience at three other installations, however, will be reviewed in a general manner insofar as data are available.

Perspective might be gained by considering both the type of surface water prevalent in New York State and experience with treatment processes analogous to diatomaceous-earth filtration. Surface waters of the northeastern part of the United States, including New York, are relatively clear, with the exception of a few large streams; they include many large natural lakes or impounding reservoirs with relatively small tributary streams. This situation led to pioneer development of slow sand filters in New York State, and 32 such plants are still in operation, including several small plants constructed in recent years.

Because of these favorable raw-water characteristics and in the light of data on diatomaceous-earth filters secured from the US Army Corps of Engrs. during World War II, the New York State Dept. of Health was interested from the first in the use of this method. Such filters provide a means of treating relatively clear water without prior coagulation and sedimentation, and thus resemble slow sand filters in their applicability under these favorable conditions.

Accordingly, the department was receptive to the plans submitted by officials of Cherry Valley, N.Y., for a full-scale diatomaceous-earth filter plant. The contractor furnishing the equipment was willing to give a year's guarantee of its effectiveness, replacing parts that proved faulty. The plans were approved, and the confidence shown by those concerned has been justified by subsequent experience in the operation of the Cherry Valley plant, as well as of similar plants at Tupper Lake and Willsboro, N.Y. A fourth installation was made at Gasport, N.Y., but very limited information is available on this unit because it was installed to clarify a chlorinated creek supply that serves as an auxiliary to the regular well supply.

Filter Structures

So far, diatomaceous-earth filters used in New York State have been of the pressure type, with the exception of several gravity units at swimming pools. In the latter type, the filter elements project upward from the bottom of open rectangular tanks, the suction pipe of the recirculating pump being connected directly to the elements, so that the gravity head of approximately 6 ft is increased by the suction head. Such open units facilitate inspection of the elements and periodic hosing to remove any adhered material, but their use in New York has been too limited to permit further conclusions.

The elements of all four diatomaceous-earth filter installations treating public water supplies in New York State are porous carborundum tubes. Some wire-wound, perforated-metal, and—more recently—cloth elements have been used at swimming pools.

Serious corrosion of the metal elements has occurred at Washington, D.C., but no instances in New York have been brought to the attention of the department. The elements of the Cherry Valley installation became partly clogged with iron deposits when the filters were first placed in use. Cleaning by soaking the elements in hydrochloric acid and sodium bisulfide semiannually, or sometimes more frequently, has permitted the continued use of all but a few of the elements. It is evident, therefore, that raw waters containing soluble iron and manganese must be appraised to determine whether prior coagulation is necessary, whether the iron is precipitated before contact with the diatomaceous earth, or whether the material remains in solution and passes through the earth and the elements.

Filter Operation

Most of the precoat applied to the elements at the beginning of each filtering cycle will fall from the vertical surfaces of the elements when filtration is stopped. If filtration again is resumed with the original precoat, some of the material filtered from the water during the previous cycle will reach the elements before the precoat is well formed, thus creating conditions favorable for the clogging of the elements. Therefore, two of the installations in New York, where automatic operation is provided, are equipped with small recirculating pumps that produce a rate of filtration somewhat in excess of 0.12 gpm per square foot of filter area. This low rate is sufficient to cause the precoat to adhere to the elements.

Although the bulk of the precoat and body feed falls from the filter elements when the filtration process is stopped, a short period of vigorous backwashing is necessary to remove all of the spent material from the elements at the end of the filtering cycle. Water for this purpose is secured from the distribution system at the plants under discussion.

Previous research by the Corps of Engineers has indicated that diatomaceous-earth filters are effective in bacterial removal even with higher than usual rates of filtration. High rates, however, lead to short filter runs because of the rapid increase in loss of head. Low rates of filtration, on the other hand, greatly prolong filter runs and, hence, increase the volume of water filtered per pound of precoat. Obviously, larger units are necessary for low rates of filtration. The design problem, therefore, is to select a rate of filtration that will balance the overall cost of diatomaceous earth and power against capital expense. Although no precise information is available for the installations in New York, it is pertinent to note that all four of them are designed to operate at low rates—between 0.50 and 0.89 gpm per square foot of filter area.

Some difficulty has been experienced in the operation of hypochlorinators as diatomaceous-earth slurry feeders. Such trouble, however, is usually associated with mechanical equipment of this type and should be avoidable by proper supervision.

The quantity of material used as a precoat usually is given as 2 oz per square foot of area of the elements. The precoat at Cherry Valley, however, has averaged 0.87 oz per square foot, which is equivalent to an average of 45 lb per million gallons of water treated. During 1952 the body feed at Cherry Valley averaged 224 lb per million gallons, but the body feed at Tupper Lake was considerably higher —560 lb per million gallons.

Although conclusions cannot be reached from the limited data available, experience in New York definitely indicates that the medium-to-coarse grade of diatomaceous earth should be used for public water supply treatment, but the grade is best selected in the light of the specific characteristics of the raw water. It is also necessary, in choosing the size and type of material, to consider the filtration rate and operating conditions, in order to achieve the longest filter runs—and, thus, the lowest amount of diatomaceous earth per unit volume of water treated—consistent with the quality of the effluent.

TABLE 1

Diatomaceous-Earth Filter Installations *

Item	Cherry Valley	Willsboro	Tupper Lake	Gasport
Date Installed	1949	1952	1952	1949
Population	760	838	5,441	800
Water consumption—*mil gal*	0.164†	0.060	0.921	0.050
No. of units	2	1	4	1
Total filter area—*sq ft*	252	115	615	63
Filtration rate—*gpm/sq ft*	0.50	0.89	0.57	0.56
Length of filter run—*hr*	48		60	36
Precoat—*oz/sq ft*	0.87			
Precoat—*lb/mil gal*	45			
Body feed—*lb/mil gal*	224†		560	
Turbidity—*ppm*				
Raw	5	trace	trace	25–40
Filtered	0.2	trace	trace	2–15
Color—*ppm*				
Raw	10	5	20	45–60
Filtered	6	5	15	20–45
Iron—*ppm*				
Raw	0.25	0.15	0.25	0.4–10
Filtered	0.15	0.40	0.20	0.10–0.60
Amorphous matter‡				
Raw	2,450			
Filtered	74			

* Prechlorination is practiced at all plants.
† 1952 average.
‡ Standard units per milliliter. Results of 32 analyses.

Filter Effectiveness

Table 1 indicates generally satisfactory removal of turbidity from the waters in question. It is difficult, from the meager information available, to determine the upper limit of turbidity that can be effectively and economically removed by these filters without prior coagulation and sedimentation. The indications are, however, that turbidities below 20–30 ppm can be removed economically without pretreatment.

On the other hand, the data definitely show that filtration through such units will not be effective in the removal of high concentrations of color. The removal of 40–50 per cent of the color concentration may be expected under favorable conditions. Whether pretreatment is necessary will depend upon the desired color of the filtered water. As consumers now are demanding clear, colorless water, coagulation will ordinarily be required when

the color of the effluent would otherwise exceed 5 ppm.

It was anticipated that prechlorination would bleach some of the color and lead to the precipitation of color complexes containing iron through the process of oxidation. The units under discussion demonstrated that prechlorination may be beneficial in this connection when the color of the raw water is moderate, but experimentation is necessary to establish the effectiveness of prechlorination for each water.

As noted previously, iron and manganese present a problem that may have to be studied through the operation of a pilot unit. The iron may occur in a form readily oxidized by prechlorination and, hence, removable by the filtering action. In other instances, colloidal iron may pass through the diatomaceous earth and be precipitated on the carborundum or metal elements. Again, the iron and manganese may remain in solution and pass through the filters without presenting any difficulty but, of course, without being removed. Additional analytical data on this subject will have to be obtained from full-scale installations or pilot plants.

Living algae will form a slimy film on the diatomaceous earth unless a proper amount of body feed is maintained throughout the filtering process. Prechlorination usually is effective in killing the organisms, which thereafter are more readily removed with other amorphous matter. The removal of algae and amorphous matter from otherwise relatively clear waters of low color would seem to represent the most fruitful field for the use of diatomaceous-earth filters.

Power and Chemical Costs

The loss of head through diatomaceous-earth filters considerably exceeds that through conventional sand filters. Therefore, the saving in pretreatment is partly reduced by the increased power needed to force the water through the units. It appears economical to use enough power so that filter runs can be extended until the loss of head reaches approximately 35 psi. Above this pressure, the loss of head seems to increase so rapidly that the gain in length of filter run is not commensurate with the power required. The ability of the filter diaphragm to withstand the pressure differential must also be considered.

The cost of the precoat plus the body feed at the Cherry Valley and Tupper Lake installations, for which significant data are available, has exceeded the cost of the alum that would otherwise have been used to coagulate the water prior to sand filtration. It appears, however, that the cost of diatomaceous earth would not exceed the cost of coagulants plus the capital charges and the expense of maintenance and operation of flocculation and sedimentation basins required for coagulation. For a proposed installation on a specific water supply, a pilot plant would be required to establish adequate cost data.

It is evident that much more detailed experience and plant operating data are needed to aid in the design and operation of diatomaceous-earth filters. Continued research must be carried on by manufacturers and engineering schools to supplement information now available.

Hydrogen Sulfide Problems of Small Water Systems

By Sidney W. Wells

A paper presented on Oct. 12, 1953, at the Florida Section Meeting, Miami, Fla., by Sidney W. Wells, Chemist, Bureau of San. Eng., State Board of Health, Jacksonville, Fla.

IN many areas of Florida, subdivision developers are confronted by the problem of providing a satisfactory water supply, because municipal or privately owned utilities either do not exist in the immediate vicinity of the development or are unable to extend their distribution lines. Unfortunately, the developer's task is often complicated by the presence of dissolved sulfides in the major water supply aquifer, the Ocala formation.

In past years a few developers have installed water systems to serve subdivisions without securing approval of the Florida Board of Health. In some instances, they have failed to provide any treatment whatsoever for minimizing or eliminating troubles due to the use of sulfide-bearing waters. In several locations, an artesian well was connected directly to the distribution mains, and the entire system was operated under the available artesian head. As this head seldom exceeds 17 psi and is usually even lower, consumers are unlikely to obtain a sufficient volume of water, particularly in two-story residences or in areas where pipelines are too small to serve the attached homes. To add to consumer worries, sulfide-bearing water is highly objectionable to most people because of its taste and odor, as well as its widely known tendency to form insoluble black iron sulfide in contact with iron mains.

One development, involving homes in the \$20,000–\$50,000 range, was served by "sulfur" water in its raw state and only under natural head. As the load increased and system pressures declined, complaints from consumers forced the developer to install pumping facilities. Nothing was done about the hydrogen sulfide content of the water except to warn prospective homeowners that no other supply was available. Needless to say, this condition cannot be favorably represented to mortgage insurance agencies and finance firms.

Typical of the complaints received by the state health board from homeowners in this and similar developments is the following letter:

> As a resident of ———, I am appealing to you for help. The drinking water in our section is completely unpalatable because of the high amount of hydrogen sulfide. It is extremely cloudy, is hard, and has a foul odor. It is ruining all table silver in a matter of minutes; if left in the water for as little as 5 min, the silver becomes completely black. The

washbasins and bathtubs are constantly left with a brown film over them which is removed only by using a large quantity of bleaching solution. The water has already started discoloring the paint on the bathroom walls, even though we have been living here but 6 weeks. We have been informed that, unless we do something about this situation, the hydrogen sulfide will clog all our plumbing pipes, particularly the washing machine connections, and they will have to be replaced in a matter of 6 months or so.

Now, my question is, could we possibly get city water in our section? We do need it desperately, and it does seem a shame that we do not even have fit drinking water.

The "extremely cloudy" characteristic noted in the preceding letter was due to the reaction of dissolved sulfides with iron from the mains to form a suspension of iron sulfide (FeS) and pyrites (FeS_2). A similar condition may be caused by the action of bacteria in distribution mains. The black, greasy-feeling water that results impregnates laundered clothing; makes bathing in tubs or showers extremely unpleasant and unsatisfactory; stains tubs, basins, sinks, and kitchenware; and soon clogs screens or cloth filters placed on taps. Cation-exchange softeners may become clogged or the exchange mineral incrusted. Their capacity is thus reduced, and eventually complete replacement of the mineral is required.

Sulfides in well water are probably produced through chemical and bacterial changes under anaerobic conditions far underground. Sulfates may be reduced to sulfides by organic matter under anaerobic conditions, and the resultant metallic sulfide changed to hydrogen sulfide by the action of carbonic acid. Sometimes sulfide is derived from the anaerobic reduction of organic matter with which the water comes in contact (*1*). Various sulfur bacteria have the ability to produce hydrogen sulfide under anaerobic conditions, some through the reduction of sulfates, sulfites, or even sulfur, and others through the decomposition of organic material (*2*).

As the chemical test for dissolved sulfides must be initiated at the well site, very few analyses reveal the sulfide content (*1, 3*). In his travels throughout Florida, and in preparing the material for this paper, the author made numerous sulfide determinations and agrees with others that most artesian well waters in Florida contain 1.0–4.0 ppm of dissolved sulfides as H_2S.

Nordell (*3*) states that the offensive, rotten-egg odor of "sulfur waters" is noticeable even at low temperatures, when the sulfide concentration is 0.5 ppm or more. Less than 0.35 ppm, however, will cause few people to complain of its presence. Artesian well waters in Florida usually have a pH of 7.2–7.8. If the pH is raised to 9.0 or more, most of the sulfide odor will disappear, owing to conversion of the weakly ionized hydrogen sulfide to alkaline sulfides. Also, the taste of the water will be somewhat improved.

Methods of Removal

According to Pomeroy (*4, 5*), when treating water supplies to remove sulfides, the total dissolved-sulfide content must be taken into account, because hydrogen sulfide is present as long as any dissolved sulfide exists. Pomeroy has calculated, for a typical water supply at a temperature of 20°C, a series of factors showing the amount of total

sulfide existing as hydrogen sulfide at various pH values.

Table 1, based on Pomeroy's calculations, indicates that, at a pH of 5.0, for example, 98 per cent of the total dissolved sulfides exists in the form of undissociated hydrogen sulfide. At pH 7.4, only 17 per cent exists in this form. It can readily be seen that increased elimination of the dissolved gas should be obtainable by lowering the water pH before aeration. Pomeroy states that these figures indicate the relative rate at which free hydrogen sulfide gas will escape from solution. At pH 7.0, hydrogen sulfide can escape from solution only approximately one-third as fast as at pH 5.0, whereas, at pH 9.0, the amount escaping will be negligible under most conditions.

The data in Table 1 do not indicate the total amount of hydrogen sulfide that can escape, because, as fast as it is removed, the equilibrium is reestablished by the combination of H^+ and HS^- to form more hydrogen sulfide. Thus, all the sulfur in any dissolved inorganic sulfide may be ultimately removed as hydrogen sulfide.

Unless sulfides are removed from water used for human consumption, many undesirable situations requiring correction will arise. Some of these conditions have already been mentioned. Another very important one results from the corrosive nature of sulfide waters. Aside from the fact that they attack iron and steel to form "black water," sulfide waters also attack copper, copper alloys, and galvanized piping, even in the absence of oxygen.

In view of the many problems connected with the use of sulfide-bearing waters, it is imperative that this dissolved gas be removed by effective treatment in private, public, or industrial water systems. A number of methods are in common use:

1. Aerators in elevated tanks.
2. Atmospheric aerators using natural-draft ventilation, in conjunction with ground storage reservoirs. These aerators are usually of the slat tray type, with or without coke, crushed stone, or the like. Adjustable spray nozzles also are used.
3. Atmospheric aerators using forced-draft ventilation, in conjunction with ground storage reservoirs.

TABLE 1

Hydrogen Sulfide Factors

pH Value	*Factor*
5.0	0.98
5.4	0.95
5.8	0.89
6.2	0.76
6.6	0.56
7.0	0.33
7.4	0.17
7.8	0.073
8.2	0.031
8.6	0.012
9.0	0.0050
9.4	0.0020
9.8	0.00079

4. Atmospheric aerators, either natural or forced-draft, together with pH reduction by the addition of mineral acids or scrubbed flue gases.
5. Atmospheric aerators in which the water, issuing from vertical risers, flows over weirs or a series of steps into ground storage reservoirs.
6. Aeration with compressed air in hydropneumatic tanks. The air enters the tank with the water through a nozzle or jet or through a grid system within the tank. Air bleeders or air relief valves serve to remove released hydrogen sulfide and excess air.

7. Chlorination, to oxidize the sulfides to free sulfur or sulfuric acid.

8. Ion exchange (*6*).

Most Florida subdivision water systems use Methods 1, 2, 6, or 7. As many of these systems are in Duval County and very few data on their efficiency in removing sulfides are available, a study of several systems in that area was recently made. The information secured is given in Tables 2–4,* and the various processes employed are discussed below.

* The iodine–sodium thiosulfate method (*7*) of determining dissolved sulfides in water was used in all field tests. Normalities of solutions were checked after each field trip. Dissolved-oxygen determinations were made using the Rideal-Stewart modification of the Winkler method (*8*).

Aerators in Elevated Tanks

Three Duval County water systems remove sulfide with aerators installed in the top of elevated tanks. At the Lake Forest subdivision water supply, the riser pipe is carried across the tank just above the overflow point and terminates in a sealed end. Holes 1 in. in diameter on 12-in. centers extend across the upper surface of the pipe. A tee, with open end facing upward, is in the center of the cross-arm aerator. A 12-in. wide, plastic-screened opening in the tank wall at aerator level provides ventilation.

There are no metering devices in this system—a serious failing of many subdivision installations. On the basis of pump operation during the test pe-

TABLE 2

Elevated-Tank Aerators *

Item	Lake Forest		Ortega Terrace		Ponte Vedra Beach			
					North System		South System	
	Well Water	Tank Effluent	Well Water	Tank Effluent	Well Water	Tank Effluent	Well Water	Tank Effluent
Air temp.—°F	85	85	88	88	82	82	81	81
Water temp.—°F	77	78	74	76	80	80	81	81
pH	7.5	7.9	7.7	7.9	7.3	7.5	7.3	7.5
Alkalinity ($CaCO_3$)—*ppm*	140	140	114	114	126	126	142	142
CO_2—*ppm*	7.0	2.8	4.2	2.7	13.5	8.5	15.2	9.7
Odor	H_2S	H_2S†	H_2S	none	H_2S	none	H_2S	none
Taste	H_2S	H_2S†	H_2S	none	H_2S	none	H_2S	none
Appearance	clear	stringy‡	clear	clear	clear	clear	clear	clear
Sulfides (H_2S)—*ppm*	2.0	0.2	1.4	0.0	2.8	0.0	2.7	0.0
Sulfide removal—*per cent*		90		100		100		100
Detention period—*hr*		4½		2¾	24+	24+	24+	24+
DO—*ppm*	0.0	4.3	0.0	4.5	0.0	3.1	0.0	2.8
DO saturation—*per cent*		52		53		38		34
Well diameter—*in.*	10		8		6		6	
Well depth—*ft*	1,160		700		600		600	
Design rate—*gpm*	800		600		750		750	

* Aerator design: perforated pipe inside elevated tank. No plants were metered.
† Slight.
‡ Tank not cleaned since erection in 1947.

TABLE 3—*Atmospheric Aerators With Ground Storage Reservoirs*

Item	Arlington Manor			Arlington Manor		Venetia Terrace			Lake Lucina			Magnolia Gardens			Lake Shore Terrace		
	Well Water	Aerator Effluent	Reser-voir Effluent	Well Water	Aerator Effluent	Well Water	Aerator Effluent	Reser-voir Effluent	Well Water	Aerator Effluent	Reser-voir Effluent	Well Water	Aerator Effluent	Reser-voir Effluent	Well Water	Aerator Effluent	Reser-voir Effluent
Air temp.—°F	84	84	84	89	89	83	83	83	84	84	84	91	91	91	78	78	78
Water temp.—°F	79	79	79	78	78	78	78	79	82	82	82	77	77	79	82	82	82
pH	7.4	8.1	7.6	7.4	7.9	7.8	7.9	7.9	7.5	7.9	7.5	7.5	7.9	7.5	7.7	8.1	7.9
Alkalinity ($CaCO_3$)—*ppm*	140	140	140	140	140	110	110	110	144	144	144	138	138	138	118	118	118
Theoretical CO_2—*ppm*	8.6	1.8	5.7	8.6	3.7	3.4	2.7	2.7	9.8	3.7	9.8	7.0	2.8	7.0	4.3	1.8	2.7
Odor	H_2S	H_2S	none	H_2S	H_2S	H_2S	H_2S	none	H_2S	H_2S	none	H_2S	H_2S	none	H_2S	H_2S	none
Taste	H_2S	H_2S	Cl‡	H_2S	H_2S	H_2S	H_2S	good	H_2S	H_2S	good	H_2S	H_2S	Cl‡	H_2S	H_2S	good
Appearance	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear	clear
Sulfides (H_2S)—*ppm*	2.0	1.1	0.0	2.1	1.3	1.4	0.9	0.0	2.3	1.5	0.0	2.0	1.4	0.0	1.7	1.0	0.0
Sulfide removal—*per cent*		45.0	100.0		38.0		36.0	100.0		35.0	100.0		30.0	100.0		41.0	100.0
DO—*ppm*	0.0	5.1	5.4	0.0	3.9	0.0	5.4	5.8	0.0	4.8	5.7	0.0	2.3	4.9	0.0	4.9	5.8
DO saturation—*per cent*		62.0	66.0		47.0		65.0	70.0		61.0	72.0		27.0	60.0		62.0	73.0
Res. det. period*—*hr*		24+					9⅛			1⅞			24+			6⅛	
Plant metered		yes			yes		yes§			no			yes			no	
Well diameter—*in.*		10			10		8			12			10			10	
Well depth—*ft*		1,002			1,002		836			1,182			1,047			1,300	
Well discharge†—*gpm*																	
Artesian		105					500						640			270	
Pump					585					725							
Aerator design																	
No. of slat trays		3			3		4			4			‖			4	
Total tray area—*sq ft*		99			99		144			144						132	
Design load—*gpm/sq ft*		7			7		2.1			3.5			500			3.7	
Actual load—*gpm/sq ft*		1.0			6		3.5			5.0			640			2.0	

* Reservoir detention period (based on plant operation rate on date tests were made).
† Actual measurement.
‡ Chlorinator cut off before making H_2S and DO tests.
§ Meter-operated chlorinator only.
‖ Adjustable 8-in. nozzle.

riod, it was determined that the detention period was approximately 275 min, resulting in 90 per cent removal of sulfides. As this tank had not been cleaned in 6 years, the bowl of the tank up to the top of the effluent pipe was completely filled with precipitated sulfur, sulfur bacteria, iron sulfide, and rust, which very probably reduced the efficiency of the tank as an aeration unit. Fortunately, chlorination of tank effluent water removed all remaining sulfides.

In the Ortega Terrace–Venetia area of Duval County, the aerator in the elevated tank is similar in construction to that at Lake Forest. Tests on tank effluent water indicated 100 per cent removal of dissolved sulfides at a detention rate of approximately 160 min. Very likely, 100 per cent removal can be obtained with a somewhat shorter detention period.

Another system, at Ponte Vedra Beach, has two 50,000-gal elevated tanks, located about a mile apart, both discharging to the same distribution system. The aerator in each tank is similar to those described above, except that the holes along the top surface of the aerator pipes are approximately 1½ in. in diameter. At the base of each tank, a 750-gpm centrifugal pump picks up water from a 6-in. diameter, 600-ft deep artesian well and discharges it to the aerator. At the maximum operation rate, the theoretical detention period in each tank would be approximately 65 min, but, because it rained every day during the study, the well pumps did not operate frequently. It is, therefore, safe to assume that the detention period was 24 hr or longer. Of course, 100 per cent removal of dissolved sulfides was obtained.

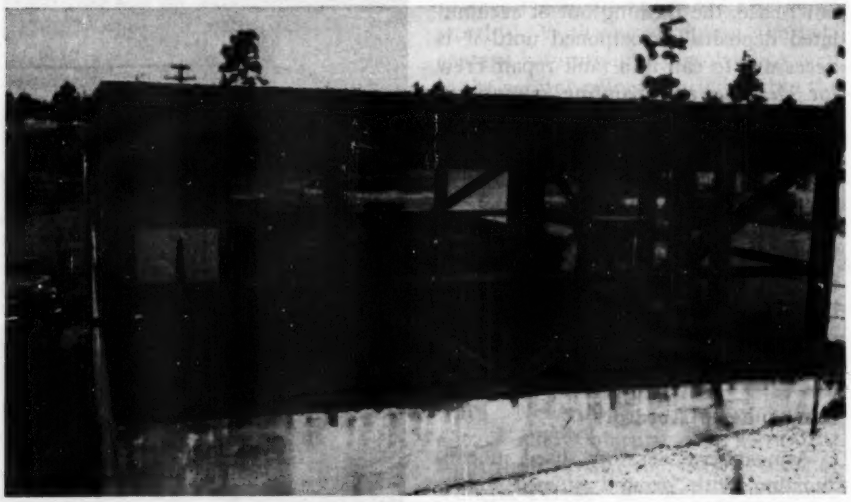

Fig. 1. Atmospheric Aerator

The screened, slat tray aerator is used in conjunction with a ground storage reservoir.

One major objection to aerators in elevated tanks is the tendency of operators to forget that such tanks are no longer merely storage reservoirs but actual chemical reaction units. Eventually, as at Lake Forest, the bowl of the tank up to the top of the effluent pipe becomes filled with sulfur products, some of which pass into the distribution system during periods of heavy demand. As none of the tanks studied is provided with a drain at the bowl base, the flushing out of accumulated deposits is postponed until it is necessary to call in a tank repair crew for cleaning and painting operations. The installation of a 3- or 4-in. drain line from the bowl to the tank leg to which the ladder is attached—where the drain should be valved—would encourage more frequent flushing of the tank. Even this arrangement might be in vain, as most operators refuse to climb elevated tanks. Perhaps the drain line should be provided at a more convenient level.

Atmospheric Aerators

Atmospheric aerators, used in conjunction with ground storage reservoirs (Fig. 1), are numerous in Duval County. Several aerators had to be omitted from the survey because they lacked entrance doors, so that collection of aerator effluent samples was impossible. As it is necessary to enter aerators for cleaning purposes, if for no other reason, doors have been ordered installed.

Table 3 reveals that sulfide removal by passage of water through the slat tray aerators alone amounted to 35–45 per cent of the total dissolved-sulfide content of the well supplies. This percentage removal was higher than expected, apparently indicating that the usual local design loading of 5–10 gpm per square foot of tray is very effective in reducing the sulfide content of artesian water. Further inspection of Table 3, however, brings out the fact that only two systems had loadings of

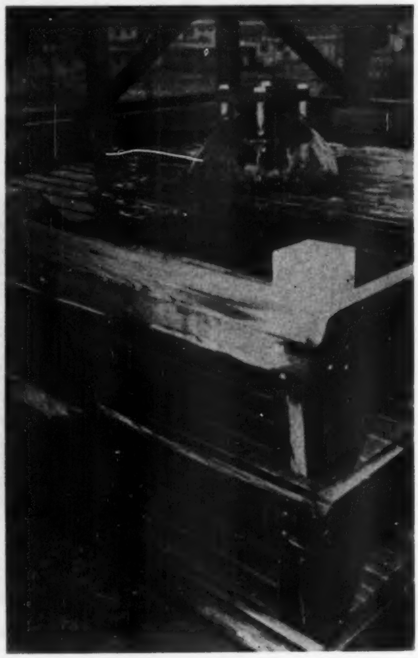

Fig. 2. Slat Trays

Artesian well water is entering the aerator at a rate of 105 gpm.

5 gpm or more per square foot with well pumps operating. Under artesian flow through the trays of the various systems studied, the loadings varied from 1 to 3.5 gpm per square foot.

Unfortunately, only one system (Lake Lucina) was operating at a flow rate high enough so that the reservoir

capacity was equaled or exceeded during the test period. In 70 min, 100 per cent removal of sulfides was obtained from the reservoir effluent water. This detention period was shorter than had been thought necessary for complete removal of sulfides, indicating the advisability of reviewing the 4-hr detention period at average flow rates used locally in designing ground storage reservoirs for oxidation of sulfides remaining after slat tray aeration. It would be presumptuous, however, to reduce design figures without further field checks. It seems desirable to run tests on one or more similar systems where reservoir effluent water could be wasted to create detention periods of 1–4 hr in order to determine, with some degree of accuracy, the minimum detention period actually needed under Florida conditions. It is hoped that this information can be secured within the next year.

The term, "reservoir detention period," used in Table 3, represents the time it would have taken water to flow through the reservoir, assuming that the rate observed during the chemical test period continued unchanged. As, in most instances, the theoretical detention period greatly exceeded the time that could be spent at any one plant, it is regrettable that higher operation rates were not in effect.

One system, that at Magnolia Gardens, uses an adjustable 8-in. diameter nozzle for aeration purposes. Sulfide removal obtained at near-maximum adjustment, avoiding spraying outside of the aerator area, was 30 per cent.

It is interesting to observe the changes in free carbon dioxide and pH recorded in Table 3. In some instances, the reservoir effluent showed a higher carbon dioxide content than the aerator effluent, a result that may have been brought about by the sulfur bacteria usually found in ground storage reservoirs. These bacteria also aid in reducing the sulfide content of influent water. In fact, the author has been informed that, at one industrial plant in Florida, these bacteria are used to remove sulfides from the artesian supply.

The results of the dissolved-oxygen determinations made on aerator and reservoir effluent water are also noteworthy. With one exception, the percentage saturation of dissolved oxygen in the aerator effluent varied within the narrow range of 61–65, whereas that in the reservoir effluent varied from 66 to 73. The exception occurred at the Arlington Manor system when tested with the well pump on.

Local slat tray aerators (Fig. 2) are generally made of 1×3-in. cypress slats with $\frac{1}{2}$–$\frac{3}{4}$-in. separations between slats and with trays 12 in. apart. The slats in one tray are designed to run perpendicular to those in adjacent trays. Nevertheless, carpenters do not always follow blueprints, as is evident from the fact that the slats in trays at Venetia Terrace and Arlington Manor are parallel.

Maintenance of open-tray aerators and ground storage reservoirs entails occasional scrubbing of trays, aerator floor, and reservoir to remove algae, bacteria, precipitated sulfur, and sometimes bloodworms (the larval stage of midge flies). Approximately 200 species of these flies are said to exist in Florida, some being small enough to penetrate 20-mesh plastic screens, so that it is difficult to keep them out of aerators and overflow pipes. Water sealing of overflow pipes is recommended for dealing with this nuisance.

Pressure Tank Aerators

For small subdivision water systems, some consulting engineers seem to feel that they cannot justify the construction of atmospheric aerators and ground storage reservoirs, and, therefore, have designed systems where aeration is performed in hydropneumatic tanks. These tanks usually vary in capacity from 1,500 to 3,000 gal, although two with a capacity of 10,000 and 12,000 gal are included in this survey. Compressed air is applied to the water in two ways: [1] through a line entering the well pump discharge pipe, either before or at the point where this pipe penetrates the base of the pressure tank; or [2] through a grid system in the base of the tank. Unoxidized hydrogen sulfide gas released in the tank, along with excess air, is bled from the tank through air relief valves or special makeshift continuous bleeders.

The usual lack of an intense hydrogen sulfide odor from air relief exhausts and the general absence of precipitated sulfur when tank drains are opened lead the author to believe that the oxidation of dissolved sulfides through the sulfur stage to alkaline sulfates occurs rather rapidly in this type of system. Operators confirm this observation.

Flentje (*9*) states that aeration by the diffusion of compressed air has unusual opportunities for application in water treatment because it can so easily and quickly be tried out and adopted. He reports that the amount of air required is 0.005–0.16 cu ft per gallon of water and that a short deten-

TABLE 4

Aeration in Hydropneumatic Tanks

Item	Saratoga Point		Roosevelt Gardens		Milmar Manor		Glynlea Park		Cedar Shores	
	Well Water	Tank Effluent	Well Water	Tank Effluent	Well Water	Tank Effluent	Well Water	Tank Effluent	Well Water	Tank Effluent
Air temp.—°F	91	91	83	83	86	86	82	82	78	78
Water temp.—°F	80	80	85	85	77	79	79	81	77	78
pH	7.5	7.5	7.6	7.7	7.5	7.6	7.3	7.4	7.6	7.8
Alkalinity ($CaCO_3$)—*ppm*	138.0	138.0	116.0	116.0	134.0	134.0	132.0	132.0	124.0	124.0
Theoretical CO_2—*ppm*	9.5	9.5	6.5	5.3	9.0	8.7	14.0	11	7.0	4.2
Odor	H_2S	H_2S†	H_2S	H_2S†	H_2S	H_2S	H_2S	H_2S‡	H_2S	H_2S‡
Taste	H_2S	H_2S†	H_2S	H_2S†	H_2S	H_2S‡	H_2S	H_2S†	H_2S	H_2S‡
Appearance	clear	milky§	clear	milky§	clear	milky‡	clear	milky§	clear	clear
Sulfides (H_2S)—*ppm*	2.0	0.3	1.5	0.3	2.0	0.6	2.0	0.7	1.4	0.9
Sulfide removal—*per cent*		85.0		80.0		70.0		65.0		35.0
DO—*ppm*	0.0	5.7	0.0	4.5	0.0	7.2	0.0	5.7	0.0	0.4
DO saturation—*per cent*		70.0		58.0		88.0		71.0		5.0
Aerator design*	*A*		*A*		*B*		*A*		*C*	
Detention period—*min*	80		22		16		50		45	
Min. pressure—*psi*	40		47		53		36		22	
Max. pressure—*psi*	50		47‖		68		36‖		43	
Air—*cfm*	3.2		6.5		2.7		3.2		3.2	
Air/water—*cu ft/gal*	0.01		0.02		0.02		0.01		0.02	
Pump capacity—*gpm*	240		400		125		400		150	
Plant metered	no		no		no		yes		yes	
Well diameter—*in.*	16		6		8		8		16	
Well depth—*ft*	995		1,118		770		812		775	

* *A*—perforated pipe in tank bottom; *B*—½-in. nipple extending into influent line entering base of tank; *C*—air enters tank with influent water.

† Very slight.

‡ Slight.

§ Owing to excess air.

‖ No change during test period; pump operated continuously.

tion period, usually approximately 15 min, is recommended. For removal of entrained gases, the higher ratio of air to water is preferred. According to Roe (*10*), more than 20-min detention is probably unnecessary, and the estimated power required averages 1.0 kw for 1-mgd capacity.

Table 4 shows that the ratio of air to water is on the order of 0.01–0.02 cu ft per gallon, a lower figure than that recommended in Flentje's report. Even with the lower ratio, the finished water was objectionably milky in at least three systems. At Roosevelt Gardens, 80 per cent removal was obtained in 22 min, while, at Milmar Manor, 70 per cent removal occurred in 16 min. With a higher ratio of air to water, more efficient removal of sulfides is likely and a shorter detention period would probably be required, but the obstacle of milky water due to dissolved air must be conquered to avoid consumer complaints and reduce corrosion of metallic mains, meters, and fittings. The greater efficiency in sulfide removal at higher pressures and with shorter detention periods is explained by the fact that the solubility of air in water increases in direct proportion to the absolute pressure. For example, the volume of air that will dissolve at 45–60-psi air pressure is three to four times as great as the volume dissolving at atmospheric pressure. Under such conditions, more oxygen is available for oxidation purposes than in atmospheric aerators, and the higher pressures tend to reduce the release of sulfides in the form of hydrogen sulfide gas.

The use of porous plates or tubes for the introduction of air from blowers has been suggested, but it is not a common practice in hydropneumatic tanks for sulfide removal. Operating under pressures of 45–60 psi, in conjunction with raw-water pumps, rapid oxidation of sulfides should occur. The disadvantages would be milky water and increased corrosion rates, although stabilization treatment could control the latter condition. Table 4 also illustrates the fact that pressure aeration is a poor way to reduce the carbon dioxide content of a water supply.

Tests on the dissolved-oxygen content of the tank effluent water were run under atmospheric pressure and, hence, Table 4 does not give the true figure for water within the tanks. The poor sulfide removal at Cedar Shores can be correlated with the low dissolved-oxygen content of its tank effluent water and with the relatively low operating pressure.

Conclusions and Recommendations

1. The reduction of dissolved sulfides in artesian water through aeration and oxidation in elevated storage tanks appear to be effective if the detention period available in the tank is on the order of 2½–3 hr.

2. Atmospheric aerators, using slat trays designed as previously described, are capable of reducing dissolved sulfides by 35–45 per cent, at least under conditions existing in Duval County, Fla.

3. Complete removal of dissolved sulfides through slat tray aerators and ground storage reservoirs, under conditions existing in Duval County, appears to be obtainable with a detention period of 1–2 hr.

4. Removal of dissolved sulfides by aeration and oxidation in hydropneumatic systems appears possible in 20 min or less under ideal conditions. The use of air diffusers (porous

media) for mixing air with water seems desirable.

5. Caution is urged in attempting to correlate zero sulfide content of plant effluent water, as found by the iodometric test method, with chlorine demand test results. Chlorine demand tests on supposedly sulfide-free water frequently yield higher chlorine demands than would be expected if complete oxidation of sulfides had occurred.

6. Aeration followed by oxidation in reservoirs should not be relied upon for complete removal of sulfides. Provision of secondary chlorination is recommended.

7. Field tests made in this study were for the purpose of determining the efficiency of the various plants in removing or reducing sulfides. It was not the object of this study to suggest changes in design data.

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Modernizing Customer Accounting and Contact at Norfolk

By R. W. Fitzgerald and Ernest C. North

A contribution to the Journal by R. W. Fitzgerald, Supt., Div. of Water Supply, Norfolk, Va., and Ernest C. North, Engr. in Charge, Valuation Dept., Whitman, Requardt & Assocs., Baltimore, Md.

THE Div. of Water Supply of the Norfolk, Va., Dept. of Public Works serves a metropolitan population of approximately 350,000. The customer contact and accounting procedures employed in the past were quite inefficient. Like many other cities of comparable size, Norfolk had a long established system of accounting, which, though adequate when installed, had been modified by patchwork changes in order to keep pace with rapid customer expansion. As a result of the extreme wartime growth in the number of customers, the "patches" cracked at the seams and the continued use of the old system could no longer be tolerated. In February 1948 the firm of Whitman, Requardt and Associates, Cons. Engrs., Baltimore, Md., presented a comprehensive report on the Norfolk water system, including an appraisal of customer accounting.

Redistricting

The first step to be taken was the redistricting of accounts in order to equalize the daily flow of work through the meter reading, billing, collecting, and accounting sections. Under the old system, the city had been divided into six billing districts, which had expanded at different rates. In 1947 the largest district had 10,000 customers and the smallest 3,800. The result was that the billings for overexpanded districts fell far behind the others, and collections and credit information were likewise in arrears. In view of the disparity in district size, it was obviously uneconomical to base the number of personnel on the needs of the larger districts. Thus, the redistricting of the customer accounts into a more flexible pattern became of paramount importance. As sufficient office workers were not available within the division to accomplish this task, specifications were prepared and public bids requested. Remington Rand, Inc., New York, was the successful bidder. The work included address verification, meter book checks, new district and route arrangement, refolioing, verifying address plate changes, preparing new meter book indexes, and compiling a street directory. This portion of the modernization program was completed in 6 months and immediately proved beneficial to the division. Much remained to be done, however, in order to modernize the system completely.

Improved Customer Contact

Private utilities had long been conscious of the value of good customer relations and had educated the public to expect and demand like service

from municipally owned utilities. Once the redistricting of accounts had smoothed out the work load on the division personnel, the next important step was the development of an "open-door" service and information section. Under the old system, the customer was required to make his contact through an old-style teller's window. Obviously, this was never the warmest or most practical type of contact, regardless of the amiability and courtesy of the clerks. The public resents the "locked-out" feeling of standing behind an opaque glass partition while the clerk scurries around the office in an attempt to gather such information as he needs to satisfy the customer.

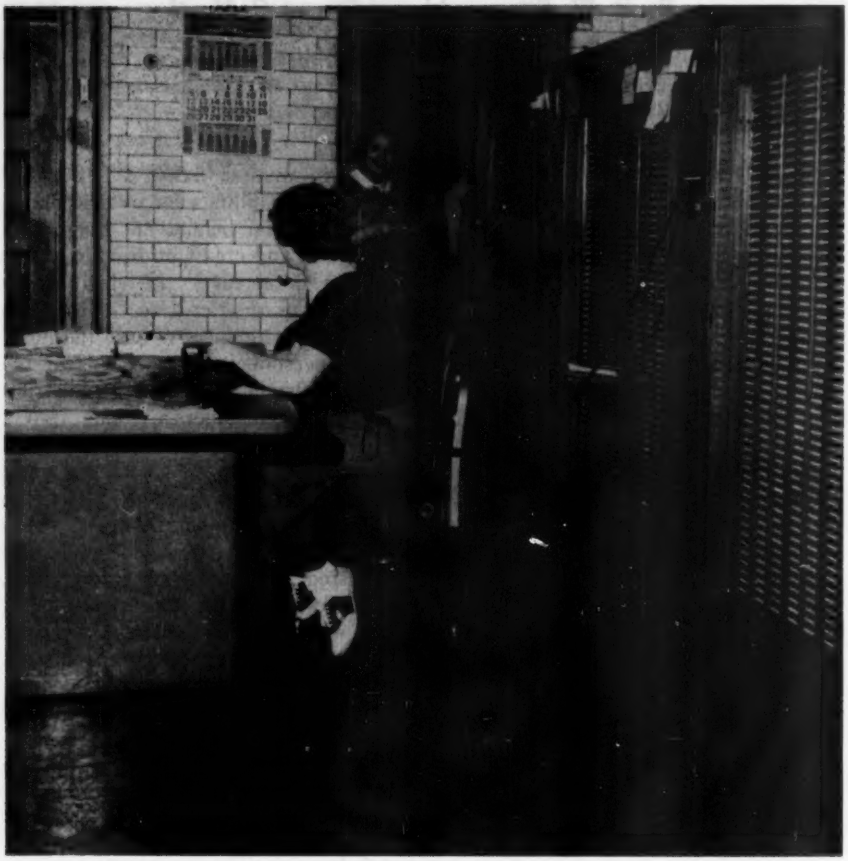


Fig. 1. File Room

The record cards in Fig. 2–4 are housed in file banks such as those shown at the right of the photograph.

The division has now taken a page from the private utilities' book, and

the public is now served by trained personnel in comfortable and pleasant surroundings. When a customer submits a problem or question on service or billing, an intercommunication system enables the contact clerk to secure promptly all of the information required. The development of the "open-door" policy, however, made it convenient to contact and posting clerks. It was, of course, necessary for the billing section, located on another floor of the building to use the ledgers also. Consequently, a great deal of confusion and loss of time was inevitable. If a contact or posting clerk needed to refer to a ledger that was being used in the billing section,

DATE — TAKEN BY	I HEREBY APPLY FOR SERVICE IN ACCORDANCE WITH THE TERMS APPEARING ON THIS AND THE REVERSE OF THIS CARD.
BUSINESS OR OCCUPATION — TELEPHONE NUMBER	
NAME OF HUSBAND OR WIFE	SIGNED:
CREDIT REFERENCE	
1.	BY:
2.	PRINT CUSTOMER'S NAME
3.	

REMARKS:

CUSTOMER HISTORY RECORD

CITY OF NORFOLK, VA. FORM NO. CA-1 NP 5108

DATE ON	ACCOUNT NO.	SERVICE ADDRESS	DATE OFF	CR. APPROVED BY

CODE	NAME	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	SPOUSE	DEPOSIT	SPEC.

Fig. 2. Customer History Record

Both the front and back of this card may be utilized to record all pertinent details of the customer's history.

necessary to install a modern bookkeeping system and a central filing system for customer service and history records.

Records and Accounting

The old method of bookkeeping was based on the so-called Boston ledger. These ledgers were kept in the office, he had to go out of the main office into the public area, walk down a long flight of stairs to the billing section, and interrupt a biller for the length of time required to search the ledger. The customer meanwhile had to stand outside the window and wait for the "traveler" to return with the information.

The system had other faults as well. After a district had been billed, the stubs from the billing section were filed by account number at a duplicate-bill window. When a duplicate was required and the account number was not known, it was necessary to refer to an address file to get the number. A cashier's stub was then prepared by hand and the customer was sent to the cashier's window for payment. The cashier sent the receipted bill stubs to the accounting section with an adding machine tape attached. A posting clerk looked up the account in the ledger, stamped the account "paid," and at the same time enumerated the paid totals on another adding machine tape. The cashier's tape and the posting clerk's tape were then reconciled. The accountant entered receipts from the posting clerk and offset these daily by cashier deposits.

DATE OF ORDER	DATE WANTED ON	TAKEN BY	CUSTOMER NAME	DATE OF ORDER	DATE WANTED OFF	TAKEN BY	REMARKS
			10				
			9				
			8				
			7				
			6				
			5				
			4				
			3				
			2				

CUSTOMER SERVICE MASTER RECORD

CITY OF NORFOLK, VA.
DIVISION OF WATER SUPPLY

FORM NO. CA-2

Fig. 3. Customer Service Master Record

This card is retained in the file at all times.

Such a crude method naturally led to many errors. It had been the custom of the department to send a delinquent notice on all bills not paid in 30 days. When the time arrived for mailing the notices, it was necessary to search the ledgers for unpaid accounts. Once again "the battle of the ledgers" had to be reenacted. Contact clerks, posting clerks, and the billing section were constantly colliding.

With the development of the "open-door" policy, the ledgers were replaced by two unit booking desks housing a modern system of stub accounting. A central information file was established, using visible-index cards (Fig. 1–4). Two sets of records are maintained, one for customer history and the other for customer service. When the information and service clerk calls the file room, he is immediately routed to the proper section, where the desired data are quickly found. The division services 50,000 active accounts, and it is obvious that considerable time is saved through this efficient system.

Billing Machines

The billing system previously used had many defects, including the in-

ability to provide useful statistics as a byproduct of the billing operation. The efficient management of a water system like Norfolk's requires such data as the water consumption by districts, the number of customers in each district, and the amount of arrears being billed by district. These statistics must be currently available, so that management can act promptly to correct any defects that may arise.

The billing machines now installed are sufficiently flexible to give the needed statistics as a simultaneous byproduct of the normal billing operation, with greater speed and accuracy than was possible with the machines and methods formerly used. The present practice is to use a postcard bill. Considerable time and money are saved, because bills are now printed by the division on a modern bill printing and addressing machine. The bills are preprinted and addressed on a cycle preparation basis.

The completely modernized system of meter reading, billing, collecting, accounting, and customer contact has now been in service for more than 2 years. Customer complaints are diminishing in number, and the irritated customer is becoming a rarity. The morale of the division's personnel is high, while the taxpayers are reaping benefits in the saving of time and money because of the improvements adopted.

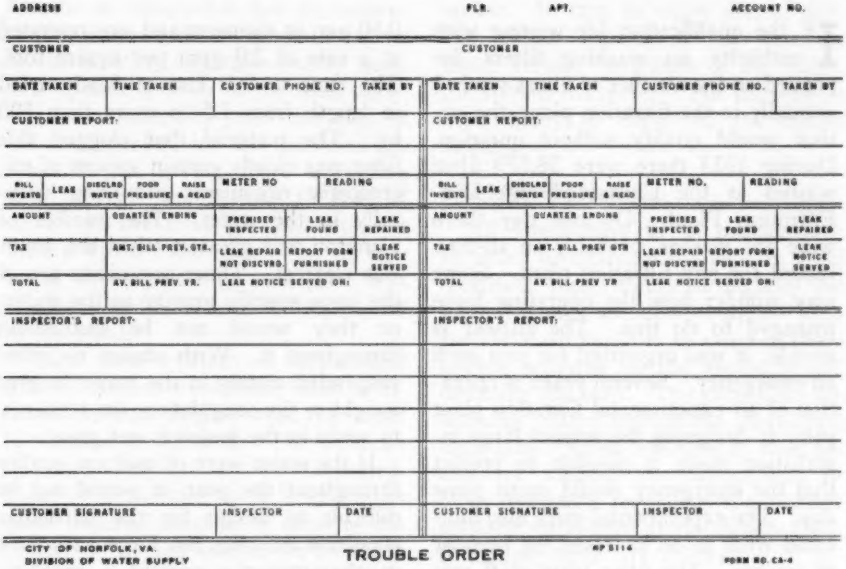

ADDRESS FLR. APT. ACCOUNT NO.
CUSTOMER
DATE TAKEN | TIME TAKEN | CUSTOMER PHONE NO. | TAKEN BY
CUSTOMER REPORT:
BILL INVESTG. | LEAK | DISCLRD WATER | POOR PRESSURE | RAISE & READ | METER NO | READING
AMOUNT | QUARTER ENDING | PREMISES INSPECTED | LEAK FOUND | LEAK REPAIRED
TAX | AMT. BILL PREV. QTR. | LEAK REPAIR NOT DISCVRD. | REPORT FORM FURNISHED | LEAK NOTICE SERVED
TOTAL | AV. BILL PREV. YR. | LEAK NOTICE SERVED ON:
INSPECTOR'S REPORT:
CUSTOMER SIGNATURE | INSPECTOR | DATE
CITY OF NORFOLK, VA.
DIVISION OF WATER SUPPLY
TROUBLE ORDER
FORM NO. CA-4

Fig. 4. Trouble Order Card

This card is taken along whenever an investigation of a service is made. Afterward the card is returned to the service file.

Washing and Maintenance of Filters

By John R. Baylis

A paper presented on Sep. 22, 1953, at the Wisconsin Section Meeting, Milwaukee, Wis., by John R. Baylis, Engr. of Water Purif., Dept. of Water & Sewers, Chicago.

IF the qualification for writing with authority on washing filters depended on the number of filters washed annually in the filtration plant, the author would qualify without question. During 1953 there were 38,579 filter washes at the Chicago South Dist. Filtration Plant. On one day there were 385 washes, probably an all-time record for any filtration plant. Some may wonder how the operating force managed to do this. The answer is simple: it was organized for just such an emergency. Several years of operation of an experimental filtration plant prior to designing the present large installation made it possible to predict that the emergency would occur some day. The experimental work also indicated what to do to lessen the number of washes. The filter plant staff was waiting for an actual occurrence to be fully convinced of the necessity of using the treatment to lengthen filter runs.

There is scarcely a plant in this country that does not have periods of short filter runs to add to other operating difficulties, although not many have such a wide range in clogging tendency as the Chicago plant. This statement was well demonstrated by operating one of the filters in the experimental plant on raw water with no coagulation and sedimentation. The filter bed was composed of sand 0.50 mm in diameter and was operated at a rate of 2.0 gpm per square foot. The runs to 8-ft loss of head varied in length from 1.5 to more than 100 hr. The material that clogged this filter was chiefly certain species of microscopic organisms occurring naturally in the water. The number of plankton was greatest when the water was clearest. These organisms are of the same specific gravity as the water, or they would not be distributed throughout it. With almost no other suspended matter in the water to give weight to the coagulation, the tendency to settle in the basins is not great.

If the water were of uniform quality throughout the year, it would not be difficult to design for the particular condition existing, but large variations in characteristics present a problem. The water being handled by the large filtration plant in Chicago is of the same quality as that in the experimental plant and is subject to the same wide range in clogging tendency.

Filter Bed Troubles

If there is a good distribution of the wash water and proper grading of the filtering materials, nearly all trouble in the filter beds may be attributed to failure of the washing system to remove from the filter all material that has been taken from the water. When a clean bed of sand is put into service

to filter water that has been coagulated and settled in the usual manner, and the filter is allowed to run until the loss of head increases to 8–9 ft before it is washed, it will be noted that some of the material filtered from the water cannot be removed except with a wash rate so high as to wash away part of the filtering material too. A portion of the mud or coagulated material has become so compacted that its suspension in water is almost the same as that of the filtering material, usually sand. The specific gravity of the mud particles is considerably less than that of the sand grains, but the particles are much larger, giving them a settling rate through water close to that of sand. If the mud particles are not broken up in some manner by an agitating force much greater than that produced by the backwash, they cannot be removed.

Each time a filter is backwashed without the aid of a surface wash, additional material filtered from the water is retained in the bed. The particles of mud become more compacted. Some of them build up to large size as more material adheres to them at the surface of the bed, or else the mudballs become pressed together so firmly during the filter run that they do not break apart at the next washing, but instead stick together to form greater masses. Usually some sand is mixed in with the mudball to give it additional weight.

The mudballs, when newly formed, tend to remain at or near the surface of the bed after washing . Eventually, however, they attain a specific gravity great enough to cause them to sink to the bottom of the sand bed during the washing period. At the same time that they are being compacted, they are being worn away gradually by abrasion of the sand during the backwashing periods. Many again become clumped together at certain points in the bed and some combine to form larger mudballs, or mud deposits, at the bottom of the sand bed or against the side wall of the filter. When the rate of wearing away is as great as the rate of formation, the volume of the clogged area in the filter does not increase. In fact, in many places, during certain seasons of the year, or under certain conditions of the water, the volume decreases. The decrease is most likely to occur in the colder months.

Washing Procedure

All filters have to be backwashed. In order to maintain the beds in good condition, many filters throughout the United States are now equipped with surface wash facilities to assist in thoroughly breaking up the mudballs and cleansing the beds. The procedure for washing filters should be developed with care. As an example of how filters are washed, the instructions to filter operators regarding the normal washing procedure at the South Dist. Filtration Plant are quoted:

The following procedure for washing filters and keeping filter log records is to be effective until further notice.

1. When to Wash a Filter

a. When the loss of head reaches 8 ft.

b. When the loss of head is approaching 8 ft on ten or more filters during short filter runs. During very short filter run periods, washing may be required before a 7-ft loss of head [is reached]. Filters with the greatest loss of head should be washed first.

c. When the filter has 48 hr of service, regardless of loss of head.

d. When the rate of flow on a fixed-rate filter (manual setting) begins to

drop and the loss of head remains constant or continues to rise.

e. When the loss of head is 6–7 ft with a low rate of flow, on a filter operating at a variable rate of flow (master control setting), and an increase in demand is expected.

2. How to Wash a Filter

a. Close 30-in. influent valve and allow the water to filter down until the water level is between the surface wash jets and the surface wash laterals. (Filters on master control, whose rate of flow has dropped almost to zero, may be "dumped." Filters that have stopped filtering because of high loss of head during short filter runs may also be dumped. Under no other conditions should filters be dumped, except by approval of filtration engineer. Record dumped filters on daily filter log. *Caution:* When dumping, do not open drain valve more than 7½ in. After dumping is complete, open drain valve fully.)

b. Close 20-in. effluent valve, open 42-in. drain valve, and turn on the electric switch to the sand expansion gage.

c. Start opening 30-in. backwash valve. When it begins to move, open 12-in. surface wash valve 6 in. Hold surface wash valve open for about 45 sec on filters in which the water level has been lowered to the surface wash jets, and about 30 sec on those in which the level cannot be lowered below the tops of the troughs. A 15-in. opening of the backwash valve will give maximum washing rate. The backwash rate is controlled by adjusting backwash rate controller to give about 3-in. sand expansion which varies with water temperature. Hold backwash valve open at maximum rate for 3 min. If, after 2 min of washing, the backwash water clears up to the extent that the surface wash laterals located about 10 in. above the sand surface are visible, the backwash valve may be closed.

d. Air binding. Filters in which the water level is allowed to go below the sand surface, and those in which the rate has fallen off because of an excessively high loss of head, may become air bound. When backwashing these filters, the air should be released slowly, in order to avoid sand eruptions. Therefore, the backwash at the beginning of the wash should be at a low rate (about 5 mgd).

3. Placing a Filter in Service

a. Close drain valve.

b. Partially open the influent valve (about 5 in.) to avoid a rapid rush of water that may disturb the sand surface. After nearly filling the filter, open the influent valve fully.

c. Open the effluent valve. Check to see that the rate controller opens up. If it does not open, check pilot valve. It may be necessary to bleed the rate controller system.

d. Turn off the electric switch for the sand expansion recorder. . . .

4. General

a. It is imperative that the instructions for filter washing procedure be followed. No deviation from these instructions is permitted without specific permission from the chief operating engineer or the filtration engineer on duty. Such deviation in washing procedure should be noted in the remarks column of the filter log sheet.

b. The procedure . . . of turning on the lights in the filter gages when a filter is ready for washing, to give visual indication to filter operators of filters needing washing, is an excellent one. . . .

c. Good filter washing operation requires that the filter washings be distributed as evenly as practical throughout the shift. Do not bunch the washes in a short period of time during the shift, for such a procedure places an excessive load on the wash water and drainage system, and indicates poor operation.

All filtration plants, with the possible exception of those operated by one man, should have a written procedure for washing filters. The procedure should meet local conditions and

need not be so specific or extensive as the one given above. The procedure should be flexible enough to take care of special conditions. To illustrate, when the filter runs are short, the washing period usually need not be as long as under normal conditions. Much depends on the rapidity with which the waste wash water clears as the backwashing continues. Some

Fig. 1. Clogged Areas and Cracks

Mounded sand is noticeable along the side wall of the filter.

plants have set a limit of turbidity—generally 10 ppm or so—for the waste wash water.

To collect a sample of water and test the turbidity while the washing is in progress would be impractical, but, by taking samples at the end of the washing periods and testing them, the operators soon become familiar with the appearance of the wash water and can guess the turbidity very well. In the South Dist. plant, the cast-iron surface wash laterals are located approximately 12 in. below the top of the wash water troughs. When the water clears so that this piping can be seen distinctly, the turbidity generally will not exceed 10–15 ppm.

Many operators continue the wash until they can see the surface of the bed clearly. In the absence of any other guide, this rule is satisfactory, although it tends to prolong the wash unnecessarily. In a number of plants, a specified period of backwashing is prescribed, and the operators do not deviate from it under ordinary conditions. Such a procedure generally requires the use of extra wash water, because the length of the period is set to meet the worst conditions. It does, however, insure that the filters will be washed thoroughly.

Clogged Areas

In the days before surface washing was adopted, many filter beds throughout the country acquired an abundance of mudballs and clogged areas. It was not uncommon to see mudballs an inch or more in diameter almost covering the entire surface of the bed. The accumulation of mudballs in certain parts of the filter soon started the formation of clogged areas, which usually deflect the backwash in such a manner as to add more mudballs to the mass.

The worst clogging occurs around the side walls. In some instances, filter beds have been pulled away from the side walls, making a crack almost to the gravel layer. This condition is caused by a slight shrinkage of the bed as the loss of head on the filter is increased. Side filtration occurs through the crack, and a layer of coagulated material that filters from the

water forms on the side, just as on the surface of the bed. When the filter is washed, this layer is pressed against the side wall of the filter and generally is only partly removed by the backwashing. Figure 1 is a photograph of clogged areas in a filter bed.

The only way to prevent side-wall clogging is to keep the sand grains so clean that the bed will not shrink during its period of service. The purpose of the surface wash is to break up the mudballs before they become sufficiently compact to settle to the bottom of the sand bed. Another fact not generally recognized by operators is that a mudball is much more likely to sink to the gravel surface with high expansion of the sand bed than with low expansion. The very high rates of backwashing used in a few filtration plants tend to defeat their objective, in that they cause the mudballs to sink to the gravel more easily. When they once settle to the gravel layer, only a backwash rate great enough to disturb the gravel will generally be able to move the mudballs. Because much has been written about mudballs and clogged areas, an extensive description of the troubles they produce and the means of preventing their formation need not be given in this paper.

Surface Washing

For years the prevailing principle in filter bed maintenance was to wash the sand very gently so that the supposed biological film on the surface of the sand grains would not be removed. This idea probably was a carryover from slow sand filtration. Now it is known that it is far better to have a clean filter bed than film-coated sand grains.

Several methods of agitating the bed had been tried in the past. For example, revolving rakes were employed to prevent clogged areas in the round wooden filters used so extensively

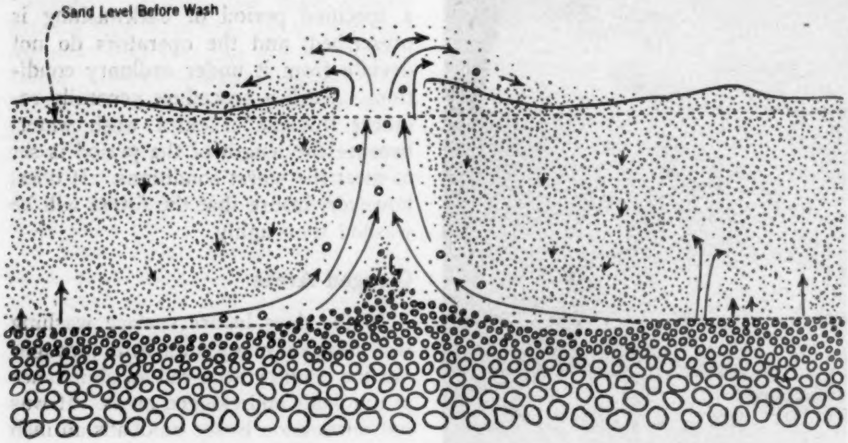


Fig. 2. Sand Boil Formation

The diagram shows a sand boil forming at the start of the wash. At the same time gravel is being pulled toward the boil and is piling up at the bottom of it.

years ago. Although there were a few early tests of surface wash jets, this method did not make much headway until after the construction of the Chicago experimental filtration plant in 1928. Surface wash jets were installed in all of the filters in that plant and proved so effective that the system was soon extended to other filtration plants. As previously mentioned, it is rapidly becoming the practice to equip all filters with surface wash facilities.

The surface wash consists of a system of pipes that expel jets of water into the sand bed at the surface in such a manner as to break up the mudballs when they start to form. Several methods of applying the jets of water have been used. A fixed jet system generally produces more rapid breaking up of the mudballs, but revolving jets have proved adequate and very helpful in many filter beds, despite the much longer period of operation generally needed. There have been so many faulty installations of homemade fixed jets that a few operators have been led to believe that they are not satisfactory. Homemade fixed jets must be installed properly if they are to give the best results.

Valve Difficulties

The backwash valves in most filtration plants become leaky after a few years of use. This statement refers to double-disc gate valves with cast-iron bodies and brass seat rings. The double-disc valve, although suitable for distribution systems, has to operate in backwash lines under conditions for which it is not basically designed. Such valves work under the highest water head of any on the filter piping. Designers of filtration plants often place the wash water tank too high. There is scarcely a filtration plant in which the backwash valves do not leak after a period of service ranging from 5 to 10 years.

Analysis of the pressure on valves during closing and opening reveals the weakness of many types. The discs are held away from the body rings by guides until ready to seat. Just before seating, the pressure of the water is transferred against the face of the downstream body ring by the disc assembly sliding off the guides. The discs then are wedged against the body rings. Opening the valve produces so much stress on the guides to raise the disc assembly that the metal is gradually crushed. The guides wear and soon allow the ring on the disc to make contact with the downstream body ring before the valve is closed. Both the disc and body rings become scarred, resulting in water leakage. The higher the head of water, the more quickly a valve will begin to scar.

So far the only remedy found has been to replace the valve, or at least the rings. Encouraging results have been obtained with a rubber-seated butterfly valve, but it has not been in service long enough for its life span to be determined conclusively. Easily replaceable valve seats and guides are the best that can now be hoped for.

Small four-way valves that operate the larger hydraulic valves have been a source of difficulty from the very beginning of rapid sand filtration and probably will continue to be troublesome so long as they are used. A four-way iron plug valve, greased at frequent intervals, may be better than the ordinary brass type, but greased valves require a separate water system—although few filtration plants have one—to avoid getting some of the grease in the water, or to discharge the hydraulic-cylinder water to waste.

If four-way valves are used, closing the pressure line to the manifold to which they are connected, after the filter has been washed, may be helpful. The four-way valves often become hard to turn with the levers provided. The leakage through such valves generally is not great and does not affect their operation.

Expansion of Sand Bed

Filter operators are aware that, as the backwash valve is turned on, the sand surface rises and a heavy concentration of coagulated material previously filtered from the water soon overflows the wash water troughs and drains out the waste pipe. Within 3–4 min the water clears sufficiently for the backwash valve to be closed. If the washing is continued a few seconds longer, the surface of the sand may be seen, first dimly and then, after 60–90 sec, very clearly. The expanded sand rises several inches above the filtering level, and moves about in a partly suspended state. In some places, boils occur, accompanied by rapid movement of the sand across the surface. Elsewhere the movement is less rapid but erratic. The speed of movement is much greater than can be accounted for by the average upward velocity of the backwash water.

Observation of the backwashing operation in a small filter with glass sides would give a better idea of what takes place in the sand bed than can be obtained by watching the surface of large filters. When the backwash valve is opened, the sand does not expand gradually, with the grains merely moving farther apart, but jets of sand and water rush upward in places at considerable velocity. Within approximately 30 sec after the backwash valve has been opened, the sand surface rises to its maximum height during the wash—3–6 in., or even more, above its level when at rest, depending on the rate of backwash.

The rate of backwash for the large filters at Chicago generally is set to produce 3–4 in. expansion of the sand bed, although most plants employ rates that result in greater expansion. The reason for the low rate at Chicago is that the surface wash is used to aid in cleansing the sand beds, making a high backwash rate unnecessary.

A backwash rate of 15 gpm per square foot of sand area would result in an expansion of 5 in. or so. At the South Dist. plant, a rate of wash just below 8 gpm is the maximum at which water can pass upward through the bed without lifting the sand. The 15-gpm rate would begin to lift the bed bodily above the gravel, with the production of a clear space between the sand and gravel, filled largely with water. Because the sand grains do not adhere, the entire bed of sand would not be lifted as a unit and remain suspended on a layer of water. Instead, the water would soon break through the bed in places, with large boils forming if the backwash valves were opened rapidly, and small boils if opened slowly. With rapid opening of the valve, the bed might be lifted 2–3 in. above the gravel before the water started to break through. Then great rushes of water and sand would be let loose at various points over the bed, usually several feet apart. Boils so great that some of the small gravel is thrown above the sand bed have been observed. Such rough handling of the gravel layers that were so carefully placed in the filter beds during construction is obviously undesirable.

Figure 2 is an attempt to show diagrammatically the formation of a sand

boil. After the bed has been lifted above the gravel layer and is floating on water, weak points develop and holes open up, permitting water with a small amount of sand to rush out. If the sand bed is lifted only a little, it will cause gravel 2–3 ft away to be pulled toward the boil, and a pile 2 in. or more above the original gravel layer will form at the bottom of the boil. Repeated production of large boils in backwashing soon causes unevenness of the gravel layer, particularly if the boil occurs at the same spot a few times. Removal of the finer gravel layer in places may allow sand to come in contact with coarser gravel, leak through it, and pass out of the filter by way of the underdrains. This is not an uncommon occurrence in many filtration plants. In fact, very few plants never have sand leakage.

Jet Action

There may be other conditions that produce boils in filter beds. If a boil persists after the bed has fully expanded, rapid turning on of the backwash valve may be basically responsible, but jet action at the sand-gravel junction also may play a part. The term "sand-gravel junction" refers to that part of the filter where the material that is in motion during backwashing joins the gravel or material not in motion. The water does not pass from the motionless gravel to the partially suspended sand at a uniform velocity. In numerous areas, the water and sand travel upward at many times the average upward velocity of the water, while, at other places, the sand travels downward, somewhat as shown in Fig. 3. The result is a series of jets of water and sand over the area, at inter-

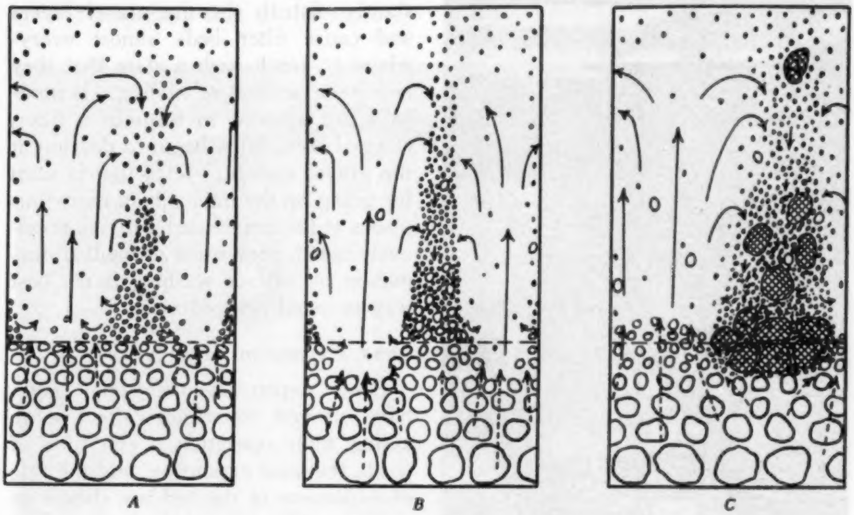


Fig. 3. Jet Action at Sand-Gravel Junction

Key: A—*newly constructed bed;* B—*condition after several washes;* C—*mudballs forming clogged area.*

vals of several inches. The velocity in the jet is so great that it will move gravel particles several times larger in diameter than the coarse sand grains at the bottom of the sand bed. With a backwash rate of 15 gpm per square foot, the average upward velocity of the water should be less than 0.1 fps.

Fig. 4. Sand Expansion Gage

The float rides the surface of the expanded sand. The pneumatic transmitter sends the signal to a recorder located elsewhere.

The velocity within the jets, however, appears to exceed 1.0 fps and is thus more than ten times the average upward velocity.

Jet action is a natural phenomenon that will take place wherever water flows upward through porous material with a velocity causing that part of the material to be suspended. It is not confined to situations involving sand and gravel, but will also be produced where fine sand overlies coarse sand and the upward velocity suspends only the fine sand. The size and specific gravity of the material merely affect the velocity, size, and length of the jets, not their formation. Jet action can be observed in glass-enclosed or glass tube filters.

These jets of water and sand constantly disturb the fine-gravel layers and cause filter beds almost everywhere to reach such a state that they require occasional rebuilding. If mudballs are allowed to form in a filter, clogged areas often begin to develop at the gravel surface. Jet action is ideal for piling up the mudballs in numerous places at the gravel surface. As previously noted, prevention of mudball formation by surface washing is the best way to avoid clogged areas.

Sand Expansion Gage

Sand expansion recording gages have proved extremely effective in aiding filter operation. The time of wash, the sand expansion, and the rate of expansion of the bed are shown on a chart. This information provides an excellent check on how the filter operator is doing his job.

The instrument consists of a float riding on the surface of the expanded sand, a pneumatic transmitter, and a recorder. Figure 4 shows the float and transmitter arrangement.

Specific Gravity

The specific gravity of the smaller sizes of gravel ($1\frac{1}{2}$–$\frac{3}{8}$ in.) should be not less than that of the sand. Otherwise, there will be a tendency for the lighter particles of gravel to mix with the

sand, thus defeating the purpose of the fine-gravel layer. Not much attention has been given to specific gravity, for it does not make a great deal of difference if the larger particles have a relatively low specific gravity. When it is recalled that jet action at the junction of the sand and gravel will move gravel at least ¼ in. in diameter, the importance of its specific gravity becomes evident. The ideal condition would be to use gravel of greater specific gravity than that of the sand.

actually the effect of low specific gravity.

Filter Appurtenances

Filters equipped with float-operated flow rate and loss-of-head gages generally require considerable attention. Cords or cables jump the pulleys or break, and have to be serviced. A pneumatic telemetering system has performed well at the South Dist. plant.

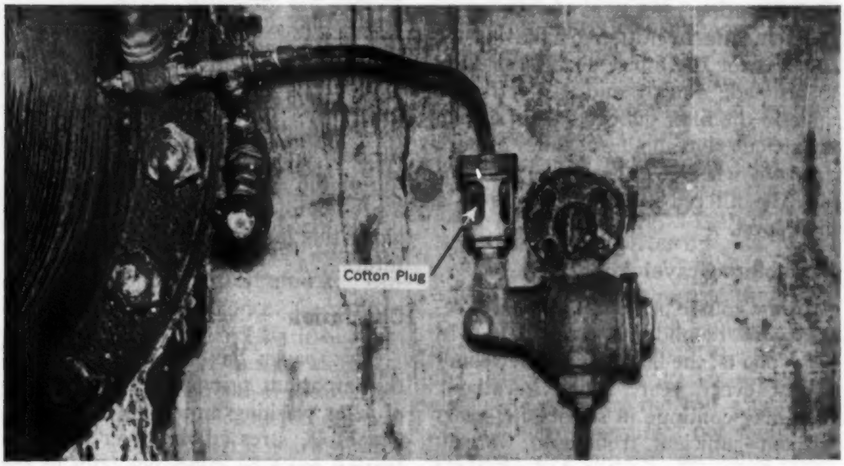


Fig. 5. Cotton Plug Filter

This device is attached to the filter effluent line.

The influence of specific gravity was demonstrated in a glass-enclosed filter in which material of approximately 2.9 sp gr was placed over ordinary gravel. After a few backwashes it could be observed that much of the fine gravel had mixed with the filtering material, which was a crushed product of ordinary sand size but more angular than sand. Probably, some of the filter bed troubles attributed to other causes are

Valve position indicators give much trouble in plants. Flexible operating shafts have proved more economical than cables or cords with counterweights.

Antiquated filter rate controllers often are a source of considerable operating difficulty, and all controllers need some servicing.

Automatic control of backwash rate is attractive in theory but may not be

practical where a wide-open backwash valve would wash sand from the filter. Failure of the control to function before the valve is open will occur often enough in some plants to cause trouble.

The cotton plug filter (Fig. 5) is a very useful device to attach to each filter effluent. Floc detectors should be employed on filters that are likely to pass flocculated material at times.

A single butterfly valve in the recovery cone of a cylindrical tube decreases the surging of the water through the filter effluent piping, particularly at high filtration rates.

Pneumatic operation of the master controls for the filters enables changes to be made quickly and easily from a table in the filter gallery. This system is particularly valuable at times of peak demand, when it is desirable to maintain the filtered-water reservoirs at a high level.

Replacing Gravel and Sand

In the course of time it becomes necessary to rebuild the filter beds. In some plants, the beds have to be rebuilt almost every year, while, in others, they may continue in service for years without sufficient trouble to warrant rebuilding. That filter beds become disturbed is partly due to the design, but mainly to the character of the water.

If a filter begins to leak sand, it should be rebuilt, unless the sand leakage is at one spot and can be corrected without removing the sand and gravel from a large area. Only a skilled operator should try to remove and replace the gravel and sand in a small area of the bed.

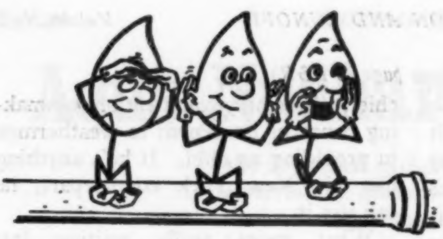
In large plants, it is cheaper to rescreen the material if the gravel and sand have become mixed. For small plants, it generally is cheaper to purchase new material, properly screened to size.

Wash Water Troughs

More study should be given to the design of wash water troughs. Heretofore they have been designed to be just large enough to carry away the wash water. If the troughs could be made to occupy a greater volume of the space above the surface of the expanded sand, the turbid material on top of the sand bed would be washed away more quickly. Developments along this line should be sought in the future.

Conclusion

This paper has discussed some of the important principles and problems of filter washing and maintenance. It should be stressed, however, that attention must be given not only to the physical and mechanical aspects but to personnel matters as well. Proper design and operation must be combined with employee efficiency and responsibility in order to achieve the desired result—an adequate supply of safe, palatable water for the community.



Percolation and Runoff

The bigwig whirligig of 1954 has already been reported to AWWA members in all the dignified significance of Constitutional mandate, but though the 48 hours that followed Sunday noon, January 17, were fuller than those of any meeting ever, at least a few moments of less than major moment were managed.

First break came on Sunday evening, when a recess for dinner with the ladies provided an opportunity to listen to something not only different, but higher pitched. And after the eating, there was Finance Committee Chairman Bill Orchard, with his version of the numbers racket, to collect $76 and 76 predictions of Seattle attendance—winner take all; there was Floridian Caesar Wertz, bearing three cases of tremendous tangeloes, to trade traditional insults with Californian Larry Grayson, who brought floral invitations to a 1958 Los Angeles Convention; and there was Vic Weir to pay off his last year's bad bet of an AWWA deficit with a charming flute and a charmable snake apiece for "taker" Orchard and "coconniver" Harry Jordan. And then, after Vic had been presented a "Mystic Eye," to improve his future predictivity, the "Men's Room" sign was put up again, and dinner was worked off in a session that lasted even beyond the eleventh hour.

On Monday, once more, noses were close to the grindstone all day, except for a brief military ceremony at luncheon, when Chesapeake's Ed Hopkins, newly promoted colonel in the Army Sanitary Engineering Section, was called front and center by his opposite number in the Marine Corps, one slightly dog-eared sergeant, who, with punctiliousness only remotely reminiscent of "the best traditions of the service," presented appropriate, if not strictly official, shoulder insignia. That evening, with the bulk of business out of the way and with theatres, the Music Hall, sports, sightseeing, and even a sharp card game to choose among, even the biggest of the wigs let their hair down. Yet, when the roll was called on Tuesday morning, not only the previous 39, but 40—including New York's Jim Harding, who had landed in the hospital with a wrenched back on his way to the meeting—were on hand to rush through the remainder of the agenda before adjournment at 12:30, in time to attend the traditional New York Section luncheon meeting in their honor. There, with more than 300 on hand to salute them, the Board took its bow and then bowed out to report back home on what had been done and what had to be done to keep AWWA headed onward and upward.

Of course, all work and practically no play can't faze anyone who has already consented to all work and no pay!

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Rainmaking may be getting to be dull and uninteresting stuff, now that it is basking in the prestige of our text section, but, fortunately, there does still seem to be a little room for imagination in the field. Not of the legalistic type, though—in which Congress seems to be indulging in its effort to establish some basis for control. So far that has seemed more apprehension than imagination. More the type that earns $30,000 per year for two ex-Navy pilots from Medford, Ore., who for four years have been successfully sowing "goop" in the clouds over the Rogue River Valley to protect its fruit crop from hail damage. What if legislation, requiring close state supervision over all meteorological messing around, did finally deromanticize the goop into plain table salt—nonetheless effective? More the type that inspired an American perfume manufacturer to seed the clouds over Paris with carbon dioxide charged with "odoriferous essence," thus producing the first redolent rainfall—the first heaven-sent scent.

Going too far, though, and yet not far enough, were a group of Marine fliers back from Korea with a Maori rain dance that presumably never fails, though it produced only a drizzle in Detroit; or our Hungarian comrades who are planning to irrigate their farms with sprinklers hung from barrage balloons and fed by tractorborne pumps from canals adjacent to Hungary's thirsty fields; or, indeed, our friends at the Chesapeake and Ohio Railway who are bragging now about a customer who selected C&O tank cars in preference to cloud seeders to keep his drought-stricken factory in operation. These are the reactionaries. These, and perhaps with them A. W. (All Wet?) Anderson, San Diego weather bureau chief, whose only comment on rainmaking concerns its benefit to weathermen in providing an alibi. If he's anything like his New York counterpart, he can use it.

What we're really waiting for, though, is the first sodium fluoride seeding. That ought to give us even more uproar than downpour.

Loss of head seems to be contagious, at least out in Cleves, Ohio, where the water system caught it from the water superintendent, who then really caught it. Jerome Morris, the water superintendent as well as the complete staff of the one-man department, didn't like his long hours, didn't like his small pay, didn't like the rigors that the drought imposed on him, didn't even like the way he was being treated by his staff. So when budget time came and went without even a raise, he lost his head completely and charged out into the night to open the valves on four strategically located fire hydrants. By morning it was the system that had lost its head, all 183,000 gallons of the system storage going down the drain. And it wasn't much later that the department lost its head, the mayor firing him on his way down to help the town clerk man the pumps.

It's only fortunate, of course, that nothing more than a little discomfort to the town's 1,981 citizens resulted—a fire, for instance, could have made the vindictiveness disastrous. Inasmuch as this was the end of Cleves' second superintendent in three months—the previous one having departed without notice but with $4,300 in water funds—it is perhaps not out of order to suggest a little more appreciation of the importance of the job in the selection of the new superintendent. Heads up!

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(*Continued from page 34 P&R*)

Still another mountain is moving Mohammedward these days—and in the pipe industry no less. Prime mover is Bassons Industries Corp., producers of plastic pipe, who have put their factory on two trailer trucks that can produce the required pipe practically in the customer's trench at a tremendous saving in transportation costs. At the moment, after only four or five months of operation on this basis, Bassons' trailer crews are putting out only oil pipe, but sewer lines and pipe for corrosive liquids are not far behind, and water pipe should pose no impossible problems. It may, of course, be some time yet before our cast-iron pipe foundries, or steel pipe mills, or asbestos-cement pipe manufacturers also hit the road, but having observed other trends we're not ready to dismiss anything. Meanwhile, with the price we're paying for milk delivery these days and the producer naturally mobile, when is the dairy business going to wake up?

Anthony Santandrea has been appointed district manager of Warren Foundry and Pipe Corp. for New York sales. Formerly with the Central Foundry Co., he transferred to the Warren Foundry organization early in 1952.

Eugene L. Lehr has been appointed chief of the Municipal and Rural Branch of the Div. of Sanitation in the Public Health Service, succeeding Ralph J. Van Derwerker, who left to fill the newly created post of chief sanitary engineer, U.S. Coast Guard.

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Lois Sutherland, treasurer of the Paducah, Ky., Water Works, passed away on December 31, after an extended illness. Miss Sutherland, who had been chairman of the Kentucky-ported to have appeared in public, although ailing, for perhaps the last time at the Section's meeting at Owensboro last September, but her health failed soon afterward.

Maurice V. Hegarty, vice-president and chief of operations of the Jamaica Water Supply Co., New York, died after a very brief illness on December 24, at the age of 61. He had been with the organization since 1932.

Lon D. Wright, superintendent and general manager of the Fremont, Neb., Dept. of Utilities, died on Nov. 5 after a short illness. He was a respected authority on utilities and had received his section's Fuller Award in 1952.

Charles L. Wachs, president of the E. H. Wachs Co. of Chicago, died on December 26th. He had served the company for 60 years, and was its president for 35 of them.

A Keasbey and Mattison Co. plant for the production of "Century" asbestos-cement pipe has been completed at Santa Clara, Calif., where it will serve West Coast customers. The $2,750,000 plant occupies a 26-acre site.

Work injuries in water supply utilities are being studied by the Bureau of Labor Statistics, which has sent out report forms for the year 1953. Managers of water utilities are urged to reply to the questionnaire, which has been made as simple as possible, in order to aid the industry's self-evaluation and progress in safety. In order to obtain an undistorted picture, it is essential that the forms be executed and returned even if no reportable injuries were experienced.

Those who have mislaid the form, or who failed to receive it, are requested to send for a copy of B.L.S. 2389, "Work Injuries in Water Supply Utilities, 1953," to the Bureau of Labor Statistics, Dept. of Labor, Washington 25, D.C. Respondents who indicate their interest by checking the appropriate box will be sent a copy of the report the Bureau will prepare.

A molded plastic clamp for coupling flexible plastic pipe has been developed by Carlon Products Co. to replace the steel clamps previously used. The clamp comes in two threaded sections and is used on the outside of the pipe ends in conjunction with insert fittings or adapters to make a leakproof seal.

Five Dorr Co. department heads have been made vice-presidents at a recent board election in that company. Frank H. Conover, of the Procurement and Production Dept.; Harold B. Coulter, of the Mechanical Engineering Dept.; John D. Grothe, of the Consulting Engineering Dept.; Douglas C. Reybold, controller and director; and Elliott J. Roberts, technical director in charge of research, are the men promoted.

Clarence W. Klassen has taken a three-month leave of absence from his duties as chief sanitary engineer of the Div. of Sanitary Engineering, Illinois Dept. of Public Health, in order to accept a temporary assignment in North Borneo. In his absence, William J. Downer, assistant chief sanitary engineer, will head the division.

(Continued on page 40 P&R)

(*Continued from page 38 P&R*)

Mud was in our eye twice last month, not even counting its New Year's Eve occurrence. First it was Nile mud and the successful correction of its instability as a construction material by Point Fourmen from Arthur D. Little, Inc., of Cambridge, Mass. Faced with the problem in 4,000 Egyptian villages in which the homes made of sun-dried mud brick disintegrated each rainy season, the Little men took one look at the lack of other construction materials and at the absence of available roads and decided it would be simpler to waterproof the mud than to import fieldstone. Putting their chemical heads together they developed a method of making the sun-dried bricks water-resistant—a solution said to make the Nile bluer and American taxpayers less so. Meanwhile, in Texas, the Nile blue which the Brazos River recently was becoming was apparently not becoming. At any rate, when waste products from Dow Chemical Co.'s Texas Div. plant began to clarify the stream, natives strongly objected to the change, and now Dow's busy trying to restore the Brazos to its normal unsettled state. Whether the problem is one of keeping water out of mud or mud in water, though, mud just naturally postulates water—a situation that unfortunately often seems reversible.

Donald H. Herak, field engineer at the Spokane office of the Portland Cement Assn., has been appointed district engineer.

Fluoridation's ninth birthday found Grand Rapids, Mich., joined by at least 848 other communities since it went off the deep end on January 25, 1945. The nine years since, which have brought fluoridated water to 15,705,690 people, have been anything but unexciting, for during that time not only fluoridation but antifluoridation was born and grew strong. It hasn't always been easy to stay out of the knock-down-drag-out fights that have resulted, but to date, at least, water works men have done a notable job of keeping their end of the operation out of range of the heaviest firing. Some of that heavy firing, by the way, was put in proper social focus in the November 1953 issue of *The Scientific Monthly* by a Williams College professor of religion, who participated in the local brawl at Williamstown, Mass., where the pros won the first two elections but the antis finally prevailed on the third. Free-floating fear is what Dr. Hutchison points to as the explanation, the same kind of unreasoning fear that now perils our freedoms, the same basic kind of fear that lets the laymen believe that "natural" fluorides will do the trick where "artificial" fluorides will kill him. Not necessarily evidence of this latter is the recent demand that has forced development of special freight rates for the shipment of Hereford water throughout Texas—Hereford being just naturally a "town without a toothache."

Another nine years and, if Ike has his way, those who were born with fluoridation will be able to vote as well as to speak for themselves. And with the smiles they're destined to have, they'll not be able to lose an election. So if you want your son to be President. . . .

M. Stuart Evans has been appointed eastern regional sales manager of Darling Valve & Mfg. Co. He has been with the firm since 1950, when he became district representative at Tulsa, Okla.

(*Continued on page 42 P&R*)

(Continued from page 40 P&R)

Texas has finally met its match in drydom—statistically anyway—and by India, too, which has long been acknowledged to have the wettest spot on earth in its Assam hills, recording a rainfall of 425 inches per year. Stealing Texas' dust is western India's Rajasthan, in which "there is no record of any rain ever having fallen." Of course, those are only dry statistics, not nearly as desiccatingly descriptive as one Texas minister's report that, before the recent drought ended, "Baptists were sprinkling, Methodists were using a damp wash cloth, and Presbyterians were giving rain checks." Even that, though, isn't as good proof to us as the words of Walter Prescott Webb, writing about the drought in last December's *Harper's:*

No one, to my knowledge, has pointed out that whatever factors may have made Houston great, the most important in keeping it going is its generous supply of fresh water of good quality—that quiet and unsung catalyst of industrial civilization.

From the ridiculous to the sublime, we'll take Texas any time.

P. S. Wilson of Glen Ridge, N.J., has been appointed to represent the Kitson Valve Div. of the Welsbach Corp. He will handle the Kitson line of brass goods, fittings, and tapping machines in eastern New York, northern New Jersey, and part of Connecticut.

Glenn C. Wanich, formerly with the Bethlehem, Pa., water and sewage treatment department, has joined the field force of Fischer & Porter Company's Chlorinator Div. and will cover the eastern Pennsylvania and western New Jersey territory.

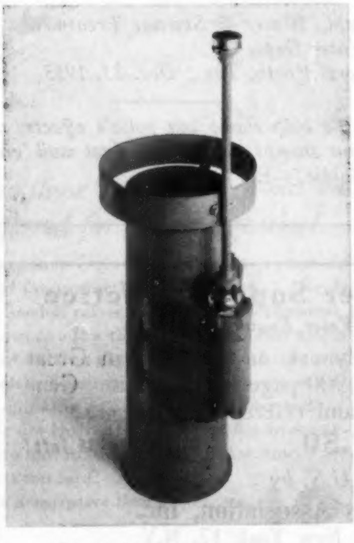
(Continued on page 43 P&R)

(Continued from page 42 P&R)

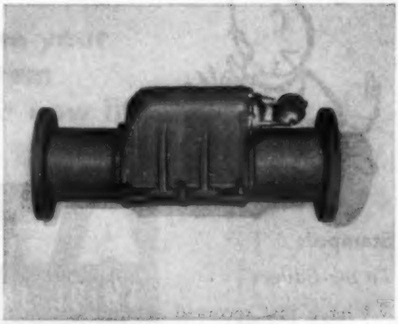
Irrigation meters have been added to the Hersey Mfg. Co. line of water meters with the development of two new types. Model MH (right) is a horizontal line meter intended for measuring flow through pipes under pressure; Model MV (below) is a vertical unit designed to measure water flowing under low head to irrigation ditches.

Both models are proportional, impeller-driven meters, with a main line and an internal orifice which shunts a definite proportion of the flow through a registering section. A conical strainer acts both to guide debris through the main section, thus protecting the impeller, and to stabilize the flow through the orifice.

Both units are said to be simple and rugged in design, delivering large volumes of water with low head loss and accurate registration. The meter parts



Vertical Irrigation Meter



Horizontal Line Meter

are being manufactured at the Hersey plant in South Boston, Mass., but final assembly, test, and distribution are being handled by the California plant, at 1544 Monterey Pass Rd., Monterey Park.

Correspondence

Stampede

To the Editor:

I have just received another letter and packet of stamps for the boy mentioned in the November issue of the JOURNAL [p. 56, P&R]. I turned them over to the boy's father today.

This stamp deal had me confused at first. Packages of stamps began arriving sometime ago and I couldn't understand why. I then saw the letter in the JOURNAL which explained the situation.

The boy mentioned in Don Williams' letter is Lee Hawthorne, the son of one of our engineers in the Sewer Dept. He was stricken with polio about three years ago. For a long time he had to get around in a wheel chair, but now he is able to walk a short distance with the aid of crutches. He is a freshman in high school, and has taken up stamp collecting as a hobby. They have a stamp collectors club in his school and he gets quite a lot of pleasure in swapping with the other club members.

I have also received responses from:

F. M. Veatch	Kansas City, Mo.
E. L. Filby	Kansas City, Mo.
E. W. Bacharach	Kansas City, Mo.
D. B. Williams	Brantford, Ont.
G. E. Blowal	Kirkland Lake, Ont.
E. S. Hopkins	Baltimore, Md.
R. E. Hansen	Mt. Clemens, Mich.
J. W. Irish	Peekskill, N.Y.
W. J. Downer	Springfield, Ill.
Jimmie Hammond	Houston, Texas

I had no idea Mr. Williams was going to write to you about this, but I am certainly glad he did so, as it was the means of providing a great deal of enjoyment for this boy. Incidents like this make one realize that there are still a lot of people in this world who think of others.

My sincerest thanks for making this possible.

W. S. MAHLIE

Supt., Water & Sewage Treatment
Water Dept.
Fort Worth, Tex.; Dec. 23, 1953

To help stamp out polio's effects, send your stamps to Lee too. Just mail 'em to Mahlie!—ED.

The Reading Meter

Your Water Supply. *American Water Works Assn., 521—5th Ave., New York 17, N.Y. (1953) 12 pp.; paperbound; single copies free; quantity prices on request*

This conservation guide is intended for distribution by local water utilities to their customers and should help not only in reducing excessive consumption and waste but also in improving public relations. It tells the householder how to read his water meter, whether the register reads in gallons or cubic feet (two separate versions of the booklet have been prepared to eliminate any confusion); how to detect a leaky toilet tank; how to drain sill cocks and pipes to avoid freezing; and how to replace leaky washers on both conventional and "newfangled" faucets. Tips on saving water in droughts or emergencies are included —and some will be found helpful at any time. Utilities planning to use quantities of the little booklet as envelope stuffers may have their name, address, and phone number imprinted on the back cover.

Steel Structures Painting Manual. *Vol. 1, Good Painting Practice. Joseph Bigos, ed. Steel Structures Painting Council, 4400—5th Ave., Pittsburgh 13, Pa. (1954) 423 pp.; $6*

This first of two volumes is well described as "a practical encyclopedia of economical and satisfactory painting methods . . . written from the viewpoint of paint users." The Steel Structures Painting Council, which sponsored the manual, was organized by a number of interested organizations, including AWWA, to determine and recommend the best methods for the protection of steel structures. In addition to the introductory chapters, which treat of surface preparation, inspection, paint quality and application, and shop application, there are a number of chapters of particular interest to the water works industry. These deal with the painting of steel tanks, hydraulic structures, pipelines, and water and sewage works structures. Where such AWWA specifications as C203, C204, C205, or D102 are applicable, they are referred to or described, but the rest of the text offers a valuable commentary and discussion.

The second volume, scheduled for publication shortly, will include all important paint systems of value in the industries covered, and also an indexed guide to the selection of the appropriate system for various conditions of service.

A Glossary of Terms in Nuclear Science and Technology. *National Research Council; published by American Society of Mechanical Engineers, 29 W. 39th St., New York 18, N.Y. Sec. VI, Biophysics and Radiobiology (1950) 43 pp.; paperbound; $0.60. Sec. VII, Instrumentation (1951) 33 pp.; paperbound; $1. Sec. VIII, Isotopes Separation; and Sec. IX, Metallurgy (1951) 47 pp.; paperbound; $1.20*

These glossaries—of which the few mentioned above include those most likely to interest the sanitary or water works engineer—are part of a series of nine prepared under the auspices of the National Research Council Conference on Glossary of Terms in Nuclear Science and Technology.

(*Continued on page 48 P&R*)

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Easier to Operate
Easier to Carry

New Hays Model "B" Tapping Machine.

The man in the ditch will welcome this new Hays Model "B" Tapping Machine.

Made of high strength aluminum alloy, it greatly reduces the weight, making it much easier to carry, handle and operate. Only 36 pounds to handle.

Also new is the renewable bearing bronze feed yoke insert, adding to maintenance economy.

Send for Folder No. 125-A.

- An aluminum alloy Tapping Machine which includes all design and operating features of the field proven Hays Model "B" bronze machine. Same working parts.
- Rugged but light weight.
- Fast and simple to operate.
- Requires minimum of working space.
- No disassembly to insert Corporation Stop — half turn of handle positions Stop for insertion in main.
- Complete with chest.

If you need a new or better tapping machine, write or wire for "The Man from Hays" to make an on-the-job demonstration of the new HAYS MODEL B Tapping Machine.

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The Reading Meter

(*Continued from page 46 P&R*)

Glacier Variations and Climatic Fluctuations. *H. W:son Ahlmann. American Geographic Society, Broadway at 156th St., New York 32, N.Y. (1953) 51 pp.; $2.50*

As dramatic a story of a warming climate and glacial recession is told by this scholarly monograph, constituting Series Three of the Bowman Memorial Lectures, as was attempted in more popular—and even sensational—form by Baxter (*Today's Revolution in Weather;* see October JOURNAL, p. 84, P&R). But this book has the advantage of being both responsible and reliable. What is more, it furnishes no real support to Baxter's contention that the climate will continue to warm until, presumably, Ultima Thule becomes semitropical. Dr. Ahlmann treats the present variation as but the latest (and best documented) of an endless series of fluctuations, and even cites quite sympathetically the opinion of a colleague that "the present amelioration of the climate has now come to an end, and the temperature will fall for the next 10 or 15 years, reaching a minimum between 1960 and 1965." He offers no grand predictions of his own, however, seeking rather to understand and interpret than to astonish. There is no need: the facts are themselves sufficiently astonishing.

Safety Pays the Smaller Business. *National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill. (1951) 24 pp.; paperbound; single copies free (quantity prices available)*

This attractive pamphlet offers convincing arguments for the dollars-and-cents *value* of safety to the smaller business, particularly in pointing out how great a ratio the indirect costs of accidents (in terms of property damage, overhead loss, time lost by all employees, operating delays, etc.) often bear to the direct costs (medical and compensation). The fundamentals of an attack on the problem are also given, with tips on methods of approach, available assistance, and practical small-plant procedures. This booklet has been recommended for foremen, supervisors, and managers of water utilities by the AWWA Safety Practices Committee, particularly if the plant does not now have a working safety program.

Handbook of Accident Prevention for Business and Industry. *National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill. (1953) 93 pp.; paperbound; $1.50 (quantity prices available)*

Intended as a condensed safety guide for the part-time safety man, this well-illustrated manual discusses the elements and applications of the job, from inspections through the keeping of good accident records. The discussion of physical plant, with its emphasis upon the importance of layout, lighting, ventilation, and housekeeping, spotlights a frequently neglected source of accident hazards. Much of the discussion of specific operations and ways of meeting their hazards will not be applicable to the water works, but there is enough of general value in this little book to make it a valuable addition to the manager or supervisor's *working* library.

A Systematic Study of the Algae of Sewage Oxidation Ponds. *Paul C. Silva & George F. Papenfuss. Pub. 7, State Water Pollution Control Board, Rm. 610, 721 Capitol Ave., Sacramento 14, Calif. (1953) 35 pp.; paperbound; free (supply limited)*

The algae discussed in this report are those which supply oxygen for secondary treatment of sewage and industrial wastes in oxidation ponds. The study took account of the composition of the algal flora in these ponds, their seasonal variations, relations with the environment, and other factors.

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The Reading Meter

(*Continued from page 48 P&R*)

What to Do Now About Emergency Sanitation at Home. *Publication H-11-1, Federal Civil Defense Administration (1953); 15¢ from Government Printing Office, Washington 25, D.C.*

Thirst comes first not only in our biased eyes, but to as dry-eyed an outfit as the Federal Civil Defense Administration. Thus, in drawing up a ten-point home preparedness program, FCDA made H_2O its ABC—not just first, that is, but first, second, and third of the things to do before disaster strikes:

1. Store right now the extra drinking water that your family might need. Don't wait for an emergency. When emergency occurs, drink only water or other liquids that you *know* are safe.

2. Know how to turn off the water service valve if necessary, and learn where to get water for emergency drinking, cooking, and washing if your outside supply fails.

3. Be prepared to purify water for drinking purposes in your own home if necessary.

And going right on from there, FCDA not only says "You Must Have Safe Drinking Water," but tells you how much: at least a gallon for drinking for each member of the family and an extra gallon for each child. Provided also are all the details on storage and emergency sources, together with detailed information on such relative inconsequentials as eating and waste disposal. Good information for you to pass on to your public.

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The Reading Meter

(*Continued from page 50 P&R*)

Water Measurement Manual. *A Reclamation Manual Specialist Supplement. Bureau of Reclamation, Denver Federal Center, Denver 2, Colo. (1953) 271 pp.; $1.50 from Bureau (Att. 841) or from Government Printing Office, Washington 25, D.C.*

This highly informative handbook provides a wealth of information on the measurement of irrigation water, with sufficient detail to serve as a guide in constructing and using the various weirs and flumes. Open-channel flow is of course stressed, and the major emphasis is given to Parshall flumes, although due attention is given to those other methods which might conceivably be found useful.

Tables of discharge and flow for a host of applications are included, and this 120-page section of the Manual is also published separately, at a cost of $1.25. Those engineers and water utility personnel who are interested in the subject matter of this manual will probably find that the slight saving in the purchase price of the tables alone will not compensate for the loss of the highly concise yet thorough text.

Proceedings of the Second Southern Municipal and Industrial Waste Conference. *Dept. of San. Eng., School of Public Health, Univ. of North Carolina, Chapel Hill, N.C. (1953) 252 pp.; paperbound; $1*

This conference, sponsored by Duke Univ., North Carolina State College, and the Univ. of North Carolina, was held at Chapel Hill on Mar. 19–20, 1953. Its proceedings include papers by R. W. Haywood Jr. on new waste products from industry, a panel led by F. W. Mohlman on treating industrial wastes in municipal plants, and a symposium on appraising stream classifications by L. A. Young, R. P. Farrell, A. H. Paessler, and G. A. Rhame. Other contributors discussed specific industrial wastes and their disposition, the relation of waste disposal to plant site location, research being conducted, and related topics.

(*Continued on page 80 P&R*)

CHANGES IN MEMBERSHIP

NEW MEMBERS

Applications received Dec. 1–31, 1953

Adams, Edward F., Chief Chem. Engr., Plastic Div., Triangle Conduit & Cable Co., New Brunswick, N.J. (Jan. '54) *P*

Allen, Ocie C., Engr., Freese & Nichols, 407 Danciger Bldg., Ft. Worth, Tex. (Jan. '54) *PR*

Angas, Robert M., Sr. Partner, Robert M. Angas & Assocs., 420 Hildebrandt Bldg., Jacksonville, Fla. (Jan. '54) *PR*

Ashmore, Willard O., Civ. Engr., Water Works, 1101 Monroe Ave., Grand Rapids, Mich. (Jan. '54) *M*

Balasha, Eliezer, Civ. Engr., 18a Arlozoroff St., Haifa, Israel (Jan. '54) *PMD*

Baron, George C., Assoc. Engr., Chief of Design, Harold S. Prescott, 594 Main St., Placerville, Calif. (Jan. '54) *PR*

Bjornson, William B., Cashier, Water Dept., Box 958, Billings, Montana (Jan. '54)

Blaisdell, F. Rodwell, San. Engr., State Board of Health, 24 McIntyre Bldg., Asheville, N.C. (Jan. '54) *P*

Blakely, H. Clealan, Mgr., Picton Public Utilities, Box 360, Picton, Ont. (Jan. '54) *PMD*

Boucher, Percival L., Chief Civ. Engr., Glenfield & Kennedy Ltd., 105 Park Street, London, W.I., Eng. (Jan. '54) *PMD*

Boylan, Thomas F., Jr., Supt., Water Dept., City Hall, New Brunswick, N.J. (Jan. '54) *PM*

Brinker, Robert O., Water Supt., Milton Freewater, Ore. (Jan. '54) *MD*

Brown, Gordon, Vice Pres., Bakelite Co., Div. of Union Carbide & Carbon Corp., 30 E. 42nd St., New York 17, N.Y. (Jan. '54) *PRD*

Brown, Roy L., Asst. San. Engr., State Health Dept., San. Eng. Bureau, State Office Bldg., Richmond, Va. (Jan. '54) *P*

Buchanan, Kenneth G. G., Water Engr., Munic. Office, Malacca, Malaya (Jan. '54)

Carr, Aleck H., Repr., Koppers Products Ltd., Rm. 217, 159 Bay St., Toronto, Ont. (Jan. '54) *MR*

Cheney, Water Dept., Fred Steiner, Foreman, Cheney, Wash. (Munic. Sv. Sub. Jan. '54) *MRD*

Clark, Bert W., Jr. Chemist & Water Bacteriologist, Water Softening Plant, 43rd & Marshall St., N.E., Minneapolis, Minn. (Jan. '54) *PM*

Crutchfield, Ralph L., Supt., Board of Water Comrs., Macon, Ga. (Jan. '54) *M*

Dawson, H. King, Gen. Engr., Water Works, 1003 Locust St., Des Moines, Iowa (Jan. '54) *M*

Dedde, Charles; *see* New Castle (N.Y.)

Delabarre, Norman L., Dist. Supt., Water Dept., Wenatchee, Wash. (Jan. '54) *M*

De La Bruniere, Tech. Adviser, Pontex Corp., 31 Nassau St., New York, N.Y. (Jan. '54) *M*

Dittig, Roger G., Jr., Asst. Engr., Hazen & Sawyer, 110 E. 42nd St., New York 17, N.Y. (Jan. '54) *PMR*

Doan, Leonard, Supt., Water Works, 125 E. Goodwin, Welsh, La. (Jan. '54) *PM*

Dooley, Andrew C., Foreman, Water Works, 651—14th St., N.W., Atlanta, Ga. (Jan. '54) *M*

Dungan, W. E., Chief Plant Engr., American Viscose Corp., Roanoke, Va. (Jan. '54) *P*

England, A. R.; *see* Graham (N.C.)

Exley, John, Pres. & Supt., People's Water Co., 165 S. Main St., Phillipsburg, N.J. (Jan. '54) *M*

Fletcher, Edgar, Water Supt., Water Works, Altoona, Kan. (Jan. '54) *MRP*

Fogarty, Joseph T., Sales Repr., James B. Clow & Sons, 201–299 N. Talman Ave., Chicago, Ill. (Jan. '54)

Franklin, George C., General Council, North Carolina League of Municipalities, 1009 Raleigh Bldg., Raleigh, N.C. (Jan. '54) *M*

Galler, William, Sr. San. Engr., Water Safety Control Sec., 3300 E. Cheltenham Pl., Chicago, Ill. (Jan. '54) *P*

Gantt, Joseph I., Salesman, Lynchburg Foundry Co., Lynchburg, Va. (Jan. '54)

Garcia-Quintero, Andres, Pres., Com. Hidrologica del Valle de Mexico, Ayuntamiento No. 146, Mexico D.F., Mex. (Jan. '54) *RPD*

Goldsmith, Marjorie (Miss), Student at Univ. of Mich., City Hall, 241 W. South St., Kalamazoo, Mich. (Jr. M. Jan. '54) *P*

Graham, Town of, A. R. England, Town Mgr., Town Hall, Graham, N.C. (Corp. M. Jan. '54) *M*

Gregory Gardens County Water Dist., Oliver S. Turpin, Inspector, Box 907, Concord, Calif. (Munic. Sv. Sub. Jan. '54)

Hacker, Theodore W., Constr. Engr. & Partner, Whitman, Requardt & Assocs., 1304 St. Paul St., Baltimore 2, Md. (Jan. '54) *P*

Henderson, Oliver A., Chief Operator, Board of Water & Light Com., New Castle, Del. (Jan. '54) *MRP*

Holland, T. J., Water & Sewer Supt., Box 41, Springhill, La. (Jan. '54) *PM*

Johnson, George W., Asst. Supt., Water Works, 405 N. Oak St., Durand, Mich. (Jan. '54)

Killian, William H., Supt., Filtration Plant, Wenatchee, Wash. (Jan. '54) *MP*

Levander, Jerry J., San. Eng. Asst., Dept. of Water & Power, 207 S. Broadway, Los Angeles, Calif. (Jan. '54) *MRP*

Logan, Harry C., Water Dept., City Hall, Seal Beach, Calif. (Jan. '54) *MRD*

Lowrie, R. C., City Engr., Box 503, Lewiston, Idaho (Jan. '54) *MRP*

(*Continued on page 56 P&R*)

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Lucoff, Julius, City Engr., Ephrata, Wash. (Jan. '54) *MRPD*

Malinowski, Edmund T., Supt., Water & Light Dept., Board of Public Utility Com., Sheboygan Falls, Wis. (Jan. '54) *M*

Manson, Owen C., Gen. Mgr., Crest Public Utility Dist., 1809 Suncrest Blvd., El Cajon, Calif. (Jan. '54) *MRP*

McNair, James S., Production Consultant, Wash. Water Power Co., 825 Trent Ave., Spokane 10, Wash. (Jan. '54) *D*

Middleton, Alfred H., Supt., Municipal Utility Board, Pryor, Okla. (Jan. '54) *M*

Middleton, Bruce; *see* Somerset Mutual Water Co.

Miller, Russell E., Tech. Director, Electric Chemical Co., 8011 Franklin Blvd., Cleveland 2, Ohio (Jan. '54) *P*

Moon, Carroll L., Civ. Engr., C. L. Moon & Assocs., 130 Casa Linda Plaza, Dallas, Tex. (Jan. '54) *P*

Moore, John J., Sales Engr., 7701 Empire State Bldg., New York, N.Y. (Jan. '54)

Nash, Jack L., Supt., Water Dept., 7300 Blanchard St., Wauwatosa, Wis. (Jan. '54) *MD*

New Castle, Town of, Water District No. 1, Charles Dedde, Water Supt., Town Hall, Chappaqua, N.Y. (Munic. Sv. Sub. Jan. '54) *M*

Orford, Harold E., Acting Chairman, Dept. of Sanitation, Rutgers Univ., New Brunswick, N.J. (Jan. '54) *MRPD*

Pravitz, Douglas P., Asst. Mgr., Lakewood Water Dist., 11900 Gravelly Lake Dr., S.W., Tacoma 9, Wash. (Jan. '54) *M*

Price, Myron H., Supt., Water Dept., Coffeyville, Kans. (Jan. '54) *PMR*

Pugh, Kyle M., Supt., Water Dept., Cheney, Wash. (Jan. '54) *MRD*

Quade, Hadley A., Engr., St. Louis County Water Co., 6600 Delmar Blvd., St. Louis 5, Mo. (Jan. '54) *MD*

Reed, Verne B., Supt. & City Engr., Munic. Water Works, Box 877, Livingston, Mont. (Jan. '54) *MP*

Rosapepe, Joseph S., Account Exec., Dudley, Anderson & Yutzy, 551—5th Ave., New York 17, N.Y. (Jan. '54) *MPR*

Rudolf, Gardner W., Resident Engr., Calgon Inc., 1225—19th St., N.W., Washington, D.C. (Jan. '54) *RD*

Rynbrand, Cornelius P., Mgr., Milwood Water Dept., 123 Exchange Pl., Kalamazoo, Mich. (Jan. '54) *MP*

Samuelson, Irving L., City Engr. & Supt., Water Works, Storm Lake, Iowa (Jan. '54) *MPD*

Schaeffer, Phillip N., Civ. Engr., Ralph L. Woolpert Co., 360 W. 1st St., Dayton 2, Ohio (Jan. '54) *MP*

Schotter, J. Leo, Supt. Water Works, Milltown, Ind. (Jan. '54) *M*

Skinner, Hervey J., Vice Pres., American Conditioning Home, Inc., 11 Melcher St., Boston 10, Mass. (Jan. '54) *P*

Smith, Henry C., Sales Engr., Fischer & Porter Co., 1621 Central Ave., Albany 5, N.Y. (Jan. '54) *PD*

Solberg, George W., Water Chemist, Dept. of Water & Sewers, Lab., Box 6, Hialeah, Fla. (Jan. '54) *P*

Sollo, Frank W., Head, Div. of Chem. Eng., Swift & Co., Research Lab., U.S. Yards, Chicago 9, Ill. (Jan. '54) *P*

Somerset Mutual Water Co., Bruce Middleton, Supt., 9701 E. Belmont Ave., Bellflower, Calif. (Corp. M. Jan. '54) *MRPD*

Steiner, Fred; *see* Cheney (Wash.) Water Dept.

Stockman, Charles E., Chief of Utilities, Alaska Dist., Corps of Engrs., Seattle, Wash. (Jan. '54) *PD*

Stoeltzing, W. A., 940 Western Ave., Pittsburgh 33, Pa. (Jan. '54) *P*

Swift, Frank M., Mgr., Orange Heights Water Co., 3968 Hamner Ave., Norco, Calif. (Jan. '54) *MD*

Taylor, Fred B., Acting Supt., Water Co., 124 W. Granite St., Butte, Mont. (Jan. '54) *MP*

Tenny, Alfred M., Sr. Water Chemist, South Dist. Filtration Plant, 3300 Cheltenham Pl., Chicago 49, Ill. (Jan. '54) *P*

Thompson, W. E., San. Engr., Kenneth P. Norrie & Assocs., Realty Bldg., Spokane, Wash. (Jan. '54) *MRP*

Tison, Leon J., Prof., Univ. of Ghent, Ghent, Belgium (Jan. '54) *MRPD*

Turpin, Oliver S.; *see* Gregory Gardens

Villa, Guy, Jr., Vice Pres., Guy Villa & Sons, Inc., 1230 Raritan Rd., Clark-Rahway, N.J. (Oct. '53)

Voss, William F., Comr. Public Utilities, City Hall, Jackson, Tenn. (Jan. '54) *M*

Wade, Jeptha A., Jr., Engr. of Design, Calif. Water Service Co., 1197 Broadleaf Lane, San Jose, Calif. (Oct. '53)

Waggoner, Wm. H., Sales Repr., Neptune Meter Co., Britt, Iowa (Oct. '53)

Watson, Burl L., Jr., Gen. Mgr., Burl Watson Domestic Water Co., 13057 E. Garvey Blvd., Baldwin Park, Calif. (Jan. '54)

Whalen, Harry A., Sales Engr., 7701 Empire State Bldg., New York, N.Y. (Jan. '54)

White, Wayne E., Supervisor of Fluorine Research, Ozark-Mahoning Co., Box 449, Tulsa, Okla. (Jan. '54) *P*

Whiteville, City of, Earl J. Wooter, Supt. Public Works, Whiteville, N.C. (Corp. M. Jan. '54) *M*

Wing, Frederick, Water Chemist, San Francisco Water Dept., 968 Washington St., Apt. 17, San Francisco 8, Calif. (Oct. '53)

Wooter, Earl J.; *see* Whiteville (N.C.)

Young, Robert C., Asst. Production Mgr., Water Works, 35 N. Main, Council Bluffs, Iowa (Jan. '54) *MP*

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Condensation

Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

POLLUTION CONTROL

Water Quality. K. H. Sharpe. Eng. Cont. Rec. (Can.), **66**:6:82 (Jul. '53). Relative importance of beneficial uses depends upon economy of area and desires of inhabitants. "Good quality" depends upon use to which water put. USPHS standards outlined. Criteria adopted by International Joint Commission on Pollution of Boundary Waters and recommended by Pollution Control Board of Ontario for lake and river waters used as sources of domestic supply include: coliform MPN median 2,400 per 100 ml; phenol or phenol equivalents, avg 2 ppb, max. 5 ppb; pH, 6.7–8.5; Fe, 0.3 ppm; threshold odor number, 4; offensive unnatural color or turbidity, none; oils and floating solids, insufficient to be deleterious; toxic wastes, within safe limits; and DO, not unreasonably reduced. These limits, which are applicable following initial diln. of any waste discharge, should be attainable if such wastes: [1] contain not more than 20 ppb phenol equivalents, 17 ppm Fe and 15 ppm oils, as detd. by extn. (faint iridescence); [2] have pH between 5.5 and 10.6; and [3] have been adequately treated for removal of bacteria and odor-producing, toxic, and deoxygenating substances. More stringent requirements may sometimes be necessary. Most common sources of taste and odors outlined.—*R. E. Thompson*

Industrial Wastes. A Photosynthesis Technique for Measuring Polluted Waters Warrants Attention of Those Interested in Stream Pollution. H. R. Murdock. Ind. Eng. Chem., **45**:101A (May '53). Because lab. conditions for measuring BOD differ so greatly from conditions prevailing in polluted stream, this test is becoming more obsolete as measure of sanitary quality of stream. For classifying polluted waters in which chlorophyll-containing organisms can propagate, method preposed by Abbott and used for 4 yr at sewage disposal plant of Nottingham Corp. in England looks particularly promising. This procedure measures oxygen photosynthesized by exposure of water sample to definite quantity of daylight, as measured by hydrogen iodide actinometer. It is based on premise that principal measurable change in polluted water in which green organisms are growing is increasing ability to photosynthesize oxygen in presence of necessary nutrients. All information necessary for preparation of reagents, use of this procedure, and interpretation of results is included. This method appears to possess sufficient merit to justify careful appraisal by those interested in stream pollution from industrial wastes.—*PHEA*

The Pollution Control Problem. A. E. Berry. Munic. Util. (Can.), **91**:7:19 (Jul. '53). Pollution problem is that of providing sufficient purif. to prevent undue deterioration in qual. of receiving body of water. Primary responsibility for prevention of pollution rests with municipality and industry. When former permits connection to sewers, it must assume responsibility for what occurs thereafter. Great postwar activity in Ontario in provision of sewage works has decreased owing to high costs. Much work still necessary. Factors influencing pollution prevention activity in Ontario include program of Dept. of Health, work of International Joint Commission on Pollution of Boundary Waters, creation of Pollution Control Board and Municipal Improvement Corp. (municipalities may use credit of province to borrow for sanitary works), and support of conservation groups. International Joint Commission agreed that secondary treatment of sewage necessary before discharge into boundary waters. More attention to operation of waste treatment plants required: too easy to open bypass valve or be content with inferior results. Reduction in number of outfalls desirable. One difficulty is securing

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joint operation of sewage works of adjacent municipalities. Responsibility for treatment of trade wastes rests with industry as part of cost of producing products.—*R. E. Thompson*

The Spanish River Investigation. A. V. DeLaporte. Munic. Util. (Can.), **90**:11:36 (Nov. '52). Conditions leading to investigation outlined. Spanish R. largest stream flowing into L. Huron west of French R. In '46–'47 bleached sulfate mill placed in operation after 15-yr interruption to pulp production in area. Wastes discharged led to court decision in favor of riparian landowners which would have closed mill in '50 and made Espanola ghost town once more. To prevent this disaster, biol. and chem. studies undertaken by Research Council of Ontario. Major portion of work conducted in field lab. at Massey and remainder at Ontario Dept. of Health Expt. Sta. Avg. stream flow for most of year, except spring, 3,000 cfs, providing 1 : 100 diln. for 30-cfs mill discharge. Latter not thoroughly mixed with river water until $2\frac{1}{4}$ mi below effluent outlet. Untreated sewage of Espanola also discharged into river.

Investigation of Wood Fiber Discharge Into Spanish River. I. G. Simmonds. Munic. Util. (Can.), **90**:12:19 (Dec. '52). Biol. investigation had shown that wood fiber formed sludge on river bottom, entrapping oil, sewage sludge, etc., and creating environment favorable to growth of bacteria, fungi, and algae. Reduction of DO occurs on decompn. and, during summer, gases released buoy masses of fibrous sludge to surface to float downstream as unsightly mats. Fiber also influences types of organisms present, *Oligochaetes* being most prevalent. Small organisms on which fish life depends not favored. Plant life coated with blackened fiber. Characteristics of various types of wood fiber discussed, and adaptation of methods employed in paper mills for identification and estn. of such fiber described in some

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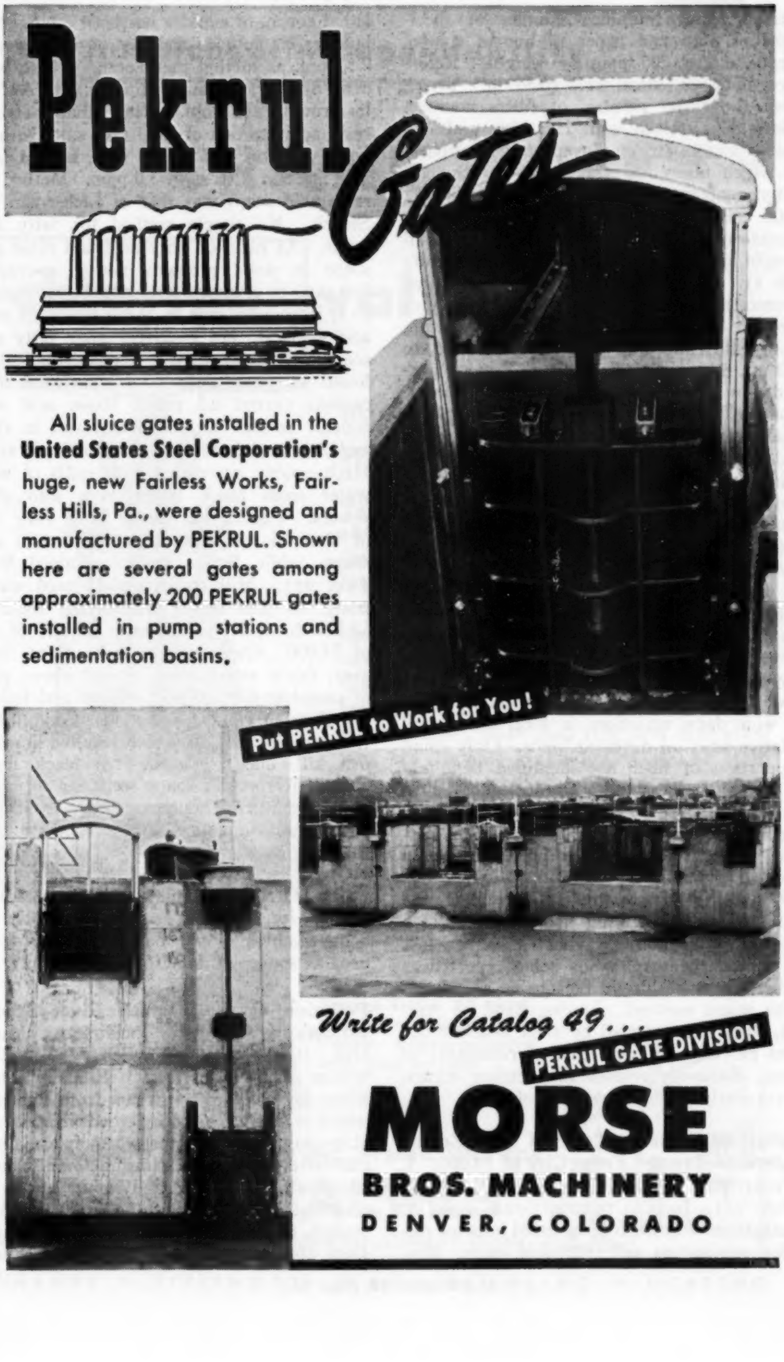
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detail. Stock reference samples of specimens of pulp and paper of known compn. prepd., contg. 200 ppm by weight. Reference slides of const. vol. (0.55 ml) prepd. from these and number of fibers counted. Slides then stained with Herzberg and "C" stains, which are employed to det. process by which fibers have been pulped. Surface samples of river water collected by dipping with calibrated pail and pouring through plankton net partially immersed in river until sample of sufficient size obtained (usually 100 l). For depth samples, force pump connected to weighted hose delivered water through baffled funnel, covered with bolting cloth, into calibrated pail. Samples dild. to 100 ml, mixed, and divided into 2 equal portions for gravimetric fiber detn. and microexamn. One portion dild. to 100 ml in graduated cylinder, mixed, allowed to settle, and sludge vol. read after 5 min and 1 hr. Slides prepd. from 0.55 ml of suspension and fiber counts made before and after treatment with Herzberg stain, latter to differentiate groundwood, coniferous, deciduous, and unclassified fibers. Avg compn. of fibers by relative number so obtained converted to avg compn. by relative weight, using appropriate factors for various types. Results shown graphically and discussed. Above Espanola, comparatively little fiber; below, fiber content of order of 1 ppm and subject to wide daily variations in total concn. and relative amts. of different kinds. Only small proportion of fiber not identified definitely as of pulp mill origin. Production of 50% H.W. (i.e., 50% poplar) pulp at mill always accompanied by increase in river fiber content. Avg weight of fiber in suspension in river below mill Oct. 10–Nov. 22, '50, was 6.97 tons per day, of which 6.69 tons of pulp mill origin, representing fiber loss of 2.48% of mill production, Gravimetric detns., loss on ignition of susp. solids being reported as "fiber," gave results in reasonable agreement with micro method. During flood of Apr. '51, susp. fiber carried by river reached 320 tons per day, 40 times that of remainder of year. Evidently considerable settling occurs, particularly in lower reaches of river.

Industrial Waste in Town of 2,400 Equals Domestic Sewage From City of 74,000. J. WILKINSON. Munic. Util. (Can.), **91**:2:22 (Feb. '53). In addn. to usual detns., chem. investigation of Spanish R. included tests for sulfides, mercaptans, and resin acid soaps. Min. lethal concn. of sulfides has been estab. at 3.0 ppm, as Na sulfide; and that of mercaptan at 0.5 ppm, as methyl mercaptan. These were detd. by potentiometric titration with $AgNO_3$ in strongly alk. soln. Min. lethal concn. of resin acid fraction of soap (Na salt) to minnows was found to be 1.0 ppm, and of fatty acid fraction (Na salt) 5.0 ppm. Method for resin acid soaps was that of Lieberman and Storch. No simple method for fatty acid soaps. As ratio of fatty acid and resin acid soaps in wood probably not in excess of 3:1, and in crude sulfate soaps usually about 1:1, it appears safe to assume that, if resin acid soaps do not exceed 1 ppm, fatty acid soaps will not exceed 5 ppm. Sulfides not found and only once (0.56 ppm) did mercaptans exceed 0.5 ppm. Resin acid soap concns. as high as 360 ppm found in river and 2 ppm 1.5 mi. beyond mouth of river. High concns. associated with spills of wash water from black liquor tank into plant sewers. Found that about 50% (dry wt) of black liquor skimmings were resin acid soaps, while liquor itself contained 0.5% (wet wt). Mill discharges 15 mgd wastes contg. 63 tons solids and having BOD of 12,500 lb, equiv. to domestic sewage of city of 74,000. Oxygen balance in river, however, fairly satisfactory, except above point of complete diln. of mill effluent and following slugs of effluent pollution. Condition of river on Nov. 23, '50, which resulted in major fish kill, due to cleaning of black liquor tanks. Resin acid soaps were far in excess of toxic limit for minnows. Even in absence of toxic constituents, O_2 depletion would have caused asphyxiation of fish. Threshold odor of river water greatly increased by mill waste. Gradually diminishes downstream but persists for 30 mi. Treatment for color and odor removal would be necessary if river used as source of domestic water supply.

Pollution of the Spanish River Studied Biologically. R. G. FERGUSON. Munic. Util. (Can.), **91**:2:31 (Feb. '53). River bottom sampled with 9" Ekman dredge. River for 20 mi. downstream from Espanola, which is characterized by wood fiber, is suitable—possibly ideal—for worm fauna almost exclusively. Other organisms never found in appreciable numbers and appeared to be transients. Above and below this 20-mi. stretch, river more suitable for diverse fauna. Data also collected on distr. of fish, aquatic

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plants, mammals and birds. Types of fish killed by 2 slugs of toxic material observed during survey discussed. In Nov. '50 fish killed as far as 32 mi. below mill. Fish caught in river had objectionable flavor, suggestive of indole and protein and resembling decaying mud. Aquatic plants scarce immediately downstream from mill; further downstream they become progressively more abundant. Domestic animals apparently unaffected by drinking river water. Considerable fiber deposits found immediately downstream from Espanola, as much as ⅓ of bottom being covered. Aquatic earthworms and lamprey larvae more abundant in that area and appeared to live in fiber beds. Spring floods clear deposits and accumulation begins again after they subside. Fiber found on lake bottom adjacent to river mouth, deposits there also contg. large pops. of aquatic earthworms. —*R. E. Thompson*

Industrial Pollution in the Cleveland, Ohio, Harbor Area. I. Physical and Chemical Results. C. C. Davis & H. B. Roney. Ohio J. Sci., 53:14 ('53). Preliminary study covering period Sep. '50–Sep. '51. Sample anals. made periodically for DO, total Fe, dissolved Fe, SO_4, turbidity, and plankton, together with field detns. of temp., pH, and Fe^{++} (qual.) at 9 points in harbor and once at 5 locations in Cuyahoga R. Data are reported in tabular form. Most severe pollution was at mouth of river inside harbor breakwater. This location generally had no DO and high values of temp., SO_4, total Fe, and suspended $Fe(OH)_3$. This area and river upstream provide unfavorable conditions for aquatic life.—*CA*

Economic Evaluation of Permanent Disposal of Radioactive Waters. A. C. Herrington, R. G. Shaver & C. W. Sorenson. Nucleonics, 11:34 (Sep. '53). Economic comparison of 5 proposed methods of concentration and permanent disposal of high-level radioactive wastes has been made. Processes were evaluated and costs estimated

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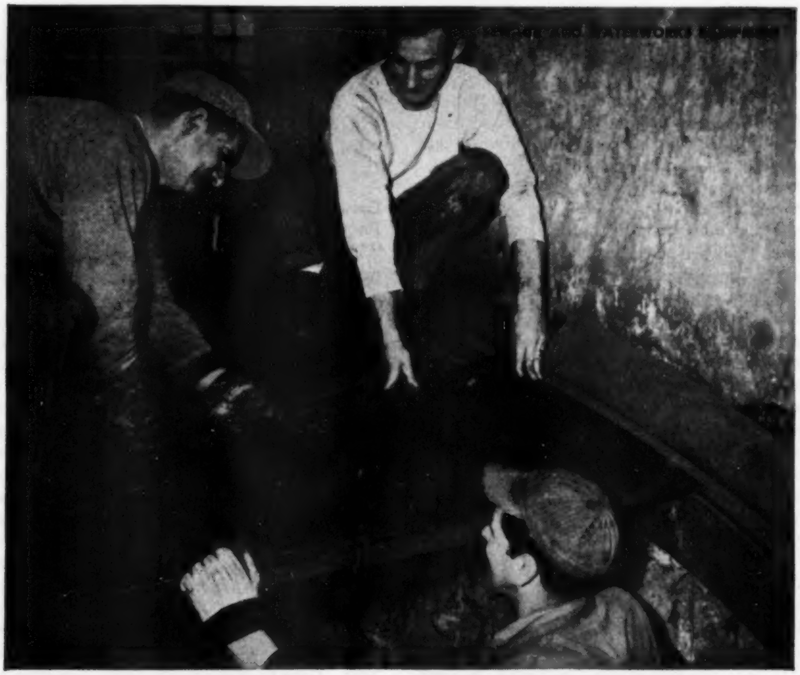

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for plant at Oak Ridge, Tenn., with assumed production rate of raw waste of 2,000 gpd at activity of 20 curies/l. Each disposal method has been broken down into series of operations: evaporation, storage, drying, etc. Pumping of liquid wastes into deep "dry" oil wells appears feasible with wells within 100 mi. Fusion of evaporated wastes into glass for land burial is less attractive than other proposed methods. Except for difficulty of leaching after burial, incorporation of concentrated liquid wastes into concrete is economical method. 3 proposals have been made for safe, permanent disposal: sea or land burial of unleached concrete, or land burial of leached concrete. Land burial without preleaching requires transportation to site having sufficiently low rainfall to preclude possibility of ground water contamination. Sea burial is practical for plants located on water navigable to ocean-going vessels. Optimum method of disposal will be dependent on location of plant with respect to dry wells, water transportation, and areas suitable for burial of unleached concrete. Cost of permanent disposal should lie in range of \$0.50–\$1.00 per initial gallon of waste, definitely competitive with present \$0.44/gal for semipermanent liquid storage. —*PHEA*

The Radium Content of Public Water Supplies. J. B. HURSH. Pub. UR-257 Univ. of Rochester, Rochester, N.Y. (May 4, '53). 27 pp. Ra concentration of source of raw water used by 41 cities in US, constituting ⅓ of total population, has been found to range from 0 to 65×10^{-16} g Ra/ml water. Of all supplies measured, only 4 showed concentrations more than 5×10^{-16} g Ra/ml. Measurement of corresponding tap water revealed range of Ra concentration (excepting one value) of 0 to 1.7×10^{-16} g Ra/ml with avg value of 0.42×10^{-16}. Exception was tap water of Joliet, Ill., measured as 58×10^{-16}. Data are regarded as generally indicating that ground water sources, deep wells in particular, contain higher Ra concentration than surface sources. Mississippi, Missouri, and Allegheny R. are exceptions to this generalization and show relatively high Ra levels. Avg value of 0.42×10^{-16} g Ra/ml tap water is not significantly different from Ra concentration in Rochester, N.Y., tap water, lending support to inference that measurements of Ra content of human subjects who lived in that vicinity may furnish values representative of much larger population group.—*PHEA*

BACTERIOLOGY

A Single-Tube Method for Anaerobic Incubation of Bacterial Cultures. W. R. LOCKHART. Science, **118**:3057:144 (Jul. 31, '53). Possible to obtain anaerobic conditions in individual tubes without special media or apparatus. Inoculate 6 × ⅝-in. culture tube contg. up to 10 ml medium, stopper with cotton plug rolled around small vial, and push down until top of vial is below tube lip. Pipet 2 ml 40% KOH and 2 ml 20% pyrogallol into vial. Immediately after adding second reagent, flame mouth of tube lightly and insert rubber stopper, previously coated with paraffin, holding firmly in place till paraffin hardens to form seal. Anaerobiosis complete and fairly rapid. Under these conditions facultative organisms do not grow on media supporting aerobic but not anaerobic growth. *Aer. aerogenes,* for example, grows aerobically but not anaerobically on synthetic medium contg. lactate as sole C source. Methylene blue indicator soln. visibly reduced in few minutes and totally decolorized in 2 hr. Anaerobic conditions accelerated by accordionwise folding strip of filter paper slightly longer than vial and placing vial on it to permit O_2 adsorption on greater surface.—*R. E. Noble*

The Biology of *Aerobacter aerogenes* (*Bacterium lactis aerogenes*) and the Intermediate Forms. W. MAASSEN, H. KNOTHE & A. T. VON KROHN. Zbl. Bakt., 1, Orig., 158, 398 ('52). Account is given of nomenclature, bacteriological classification, morphology, and properties of *Aer. aerogenes* and Intermediate group of coliform bacteria and of differentiation of these forms. Authors then describe examn. of 267 strains of forms belonging to *Aerogenes* group, derived from human sources—blood, feces, urine, etc. Methods used included Imvic series of tests (indole, methyl red, Voges-Proskauer, and citrate tests), liquefaction of gelatin, microscopic examn., tests for slime formation and its prevention, and tests of pathogenic properties. Methods and results are described and reliability of various tests for purposes of identification are discussed. Slime-forming ability of strains was found to be erratic,

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being affected mainly by temperature of environment and by number of cultures through which strain had passed. *Aer. aerogenes* strains were more constant in slime formation than *Esch. coli* or Intermediate strains. Optimum temperature for *Aer. aerogenes* was 37°C. Strains of *Aer. aerogenes* from blood grew better in slimy forms at room temperature. Certain *Esch. coli* strains were as vigorous in slime formation as *Aer. aerogenes.* Imvic test results were not always clear and some strains could not be differentiated by this method. Voges-Proskauer reaction in O'Mear's modification was found useful for differentiation. Grouping of strains according to their origin did not disclose any uniformity; it was, however, noticeable that strains from blood and duodenal liquid, which were at first slime forming, lost this capacity after several cultures and remained slimeless. Repeated culturing appeared to have effect on pathogenic properties. Classification of *Bact. pneumoniae* Friedlander is discussed.—*WPA*

Bacterial Indicators of Pollution in Surface Waters. H. V. Leininger & C. S. McCleskey. Applied Microbiology, 1:119 (May '53). In all waters studied, high total counts were associated with relatively high counts of *Esch. coli* and enterococci. Difference between relatively clean and recently polluted water was more strikingly shown by enterococci than by coliform test. 2 shallow wells included in this investigation were found to be of poorer bact. quality than most surface waters. As measured by USPHS standard, both wells were heavily polluted. Results obtained in this study indicate that value of coliform test in examn. of surface waters is limited; more reliance must be placed on sanitary inspection.—*PHEA*

A Comparative Study of Folpmers' Glutamic Acid Medium for the Detection of *Bact. coli* in Water. N. P. Burnam & C. W. Oliver. Proc. Soc. Applied Bact. (Br.), 15:1 (Apr. '52). Folpmers' glutamic acid medium and certain modifications compared

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with MacConkey's broth for detecting coli-aerogenes organisms in water. Very satisfactory results obtained with glutamic acid medium prepd. in tubes and incubated in same way as MacConkey's broth. Less satisfactory results obtained when lactose substituted for glucose in glutamic acid medium, although both superior to MacConkey's broth. Anaerobic method as described by Folpmers had too many disadvantages to be practicable for routine examn. of large numbers of samples.—*R. E. Noble*

Coli-Aerogenes Bacteria Isolated From Grass. S. B. THOMAS & J. MCQUILLIN. Proc. Soc. Applied Bact. (Br.), 15:41 (Apr. '52). Numbers of coli-aerogenes bact. on ungrazed herbage and intensively grazed pasture detd. by inoculation into MacConkey's broth incubated at 30°, 37°, and 40°C. From cultures producing acid and gas at 30°, organisms isolated on EMB agar and classified according to recommendations of British coliform subcommittee (Br.). These microorganisms abundant in grass from both habitats, sometimes exceeding 10^7/g. Numbers much lower at 37°, with 80% of ungrazed herbage samples and 37% of those from grazed pasture contg. 10^4/g. *Esch. coli*, intermediate, and aerogenes-cloacae types found with frequencies of 4, 19, and 77%, respectively, on ungrazed herbage, compared with 9, 17, and 73% on grazed pasture. Aerogenes-cloacae types dominant among the 37° positive cultures. High proportion (53%) of 249 classifiable cultures did not produce acid and gas in MacConkey's broth in 2 days at 37°. Anaerogenic and paracolon types at 30° infrequent (5%) on ungrazed herbage, but constituted 18% of classifiable cultures from grazed pasture. Unclassifiable cultures infrequent, mainly paracolons giving − − − + Imvic reactions and liquefying gelatin rapidly. No appreciable difference in incidence of different coli-aerogenes types on flowering heads of grass examd. during summer and on herbage samples examd. throughout year. Only 6 cultures from EMB plates developed golden-yellow pigment on nutrient agar characteristic of so-called *Bact. herbicola*, though lactose-negative strains producing golden- or lemon-yellow pigments were common in epiphytic microflora of herbage.—*R. E. Noble*

Some Observations on the Viability of Sewage Bacteria in Relation to Self-Purification of Mussels. H. P. SHERWOOD. Proc. Soc. Applied Bact. (Br.), 15:21 (Apr. '52). Autoclaved sea water and water removed from shell cavity of mussels supported growth of coli-aerogenes bacteria. Attempts to reproduce conditions that had favored multiplication of latter in purified mussels failed. "Enrichment" of sea water by repeated reuse for storage of successive lots of mussels, in conjunction with sand filtration and aeration under conditions equivalent to 60 wk of approved purif., did not promote growth of coli-aerogenes bacteria. Culture of *Salmonella typhi* and *S. paratyphi B* survived few days only in sea water favorable to slow multiplication of *Esch. coli* Type I.—*R. E. Noble*

Observations on the Manometric Estimation of Bactericidal Activity. W. B. HUGO. Proc. Soc. Applied Bact. (Br.), 15:29 (Apr. '52). Complexity of action of certain phenolic bactericides on metabolic activities of

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bact. suggests that interpretation of bactericidal action in terms of inhibition of metabolic process may not always be justified. Action of cetyltrimethylammonium bromide on O_2 uptake of *Esch. coli* on several substrates appears to be straightforward and correlatable with viability studies.—*R. E. Noble*

Water Pollution Research 1952. Report of the Director of Water Pollution Research. British Dept. of Scientific and Industrial Research, London, Eng. (1953). 64 pp. *Some factors affecting bactericidal action of chlorine.* Disinfectant power of chlorine is greatly modified by substances which react chemically with it. There is somewhat restricted group of chemicals with which chlorine reacts in this way when in very dilute solutions. Following substances showed chlorine demands of less than 1 ppm in concentration specified: sodium palmitate (0.005%), sodium acetate (4.0%), glycerol (0.1%), starch (0.1%), acetaldehyde (0.1%), furfural (0.1%), methyl (or ethyl) alcohol (2.0%), and glucose (2.0%). Substances with high chlorine demands were: ascorbic acid (62.5 ppm), 17–18 ppm; acetoin (0.1%), 10–11 ppm; and catechol (5 ppm) 8–9 ppm. Chlorine reacts slowly with ammonia (chloramines) and amino acids (chloro-amino products) to produce compounds with less disinfectant action than free chlorine. Addition of chlorine to solution containing bacteria and ammonia (or amino acids) has disinfectant action due first to diminishing free chlorine concentration and later to chlorine compounds which may be produced. As evidence, addition of 1 ppm chlorine to mixture of *Esch. coli* and glycine in water produces 99% kill in 22 min, while adding chlorine to glycine solution and *Esch. coli* 15 min later requires about 4 hr for bacterial kill. Chlorine added to peptone solutions up to, or slightly above, demand of peptone had no killing action in 6-hr contact period. On other hand, concentration of chlorine equivalent to small fractions of demand in sewage resulted in destruction of most bacteria in short contact periods. Surviving bacteria, however, might rapidly proliferate on prolonged standing. Further tests with peptones showed activity of chlorine was greatly increased in presence of ammonia (as ammonium sulfate) when chlorine dosage was greater than chlorine demand of peptone. Addition of 20 ppm chlorine (chlorine demand was 18–19 ppm) to peptone containing 250 ppm of ammonia resulted in over 99% kill in 6 hr. Increasing concentration of chlorine to 25 ppm and 35 ppm decreased contact period to 1 hr. Curves obtained by plotting logarithm of percentage of bacteria surviving at each time interval showed that presence of ammonia enhanced bactericidal effect of chlorine in presence of peptones. Ammonia is commonly regarded as one of substances mainly responsible for reducing bactericidal action of chlorine. In presence of certain other substances, ammonia may, in fact, increase activity of chlorine. Explanation may be not only in different bactericidal powers of various chloro-amino compounds, but also in velocity of formation and subsequent decomposition. *Enumeration of* Streptoccocus faecalis *in polluted waters.* Well defined streptococci type known as *Streptococcus faecalis* is normal inhabitant of man and other animals. It belongs to Lancefield's Group D and is characterized by heat resistance and ability to grow at high pH values and in presence of rather high concentration of sodium chloride. Fact that this organism has been used much less than *Esch. coli* as indicator of fecal pollution is due largely to lack of suitable method for detecting and enumerating it. Liquid medium containing glucose, yeast extract, and sodium azide, with incubation temperature of 45°C, has been found to be virtually specific for *Str. faecalis.* In order to make colony counts, glucose-azide agar was used. Procedure consisted of: [1] adding inoculum to melted glucose-azide agar (48°C) contained in bottle; [2] mixing; [3] placing bottle in chuck mounted horizontally, and rotating rapidly under jet of cold water; [4] incubating at 45°C (time not specified); and [5] counting colonies with aid of chamber in which bottle was illuminated through 2 longitudinal slits on under side. Young cultures were not inhibited, but procedure had very depressing effect on cell count of older cultures. Proportion of cells able to grow became less after being suspended for some time in water. This depressing effect could be overcome by technique which authors describe as "resuscitation." This consists of planting water sample in double-strength glucose-yeast extract-peptone broth, incubating 1–2 hr at 37°C, and adding broth to double-strength melted neutral red azide-agar. Remainder of procedure was as de-

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scribed above. Tests with pure cultures suspended for various periods in dilute buffer showed that, on average, if organisms were subjected to preliminary resuscitation for 2 hr, count on glucose-azide agar at 45°C approximated count on glucose-yeast extract agar (without azide) at 37°C. Further tests on natural-water samples suggested that resuscitation period should be reduced to 1 hr in order to prevent excessively high counts. Neutral red was found to have seriously depressing effect on *Str. faecalis* count which could not be neutralized by "resuscitation." It was subsequently eliminated from medium. With both *Esch. coli* and *Str. faecalis*, presence of 0.7% phosphate was found to inhibit proportion of cells. With *Str. faecalis*, it was found that, if phosphate solution was separately sterilized and added to sterile broth or agar, phosphate had no depressing effect on growth. Further tests showed that depressing effect of phosphate was due chiefly to autoclaving of phosphate with broth portion. Quantitative formulas for media were not described.—*PHEA*

A New Medium for the Detection of Enterococci in Water. W. Litsky, W. L. Mallmann & C. W. Fifield. Am. J. Pub. Health, **43**:873 ('53). Selective medium (tryptose 20.0, dextrose 15.0, NaCl 5.0, K_2HPO_4 2.7, KH_2PO_4 2.7, NaN_3 0.4, and ethyl violet 0.00012 g/l) is presented which is specific for growth of enterococci from pure cultures or from glucose-azide broth (tryptose, Difco, 2.0%, dextrose 1.50%, NaCl 0.50%, K_2HPO_4 0.27%, KH_2PO_4 0.27%, NaN_3 0.02%) showing growth from sewage-contaminated water. New test for enterococci is proposed wherein glucose azide broth is used as presumptive medium and ethyl violet-azide broth as confirmatory medium. —*CA*

The Antibiotic Power of Sea Water Against Intestinal Bacteria Discharged in Polluted Town Effluents. H. H. de Balzac et al. Bul. Acad. Nat. Med. (Fr.), **136**:514 ('52). Observations made on Mediterranean coast about time of liberation of France led to establishment of committee to study self-purification of sea water. Paper gives very brief resume of some findings and indicates lines of future research. Antibiotic power (meaning capacity to reduce numbers of intestinal bacteria) is independent of degree of salinity and diminishes from third day of storage onwards; it is destroyed by heat (autoclaving or tyndallization at 70°C) but not by Chamberland L3 filtration or exposure to sunlight; it is absent from synthetic sea water; it is not due to bacteriophage; it is increased by shaking and oxygenation; it is more active in Atlantic than in Mediterranean waters. Studies to be made include possible production of antibodies by plankton, algae, and other forms of marine life.—*BH*

Effect of Ultraviolet Irradiation on Gram Positiveness. J. W. Bartholomew & T. Mittwer. J. Bact., **63**:779 ('52). Authors found that smears from 28 species of microorganisms irreversibly lost their gram positiveness when exposed to ultraviolet light for periods up to 30 hr. Many bacterial spores which normally were unstained by Gram's stain, upon ultraviolet exposure first became Gram positive and then shifted to Gram negative after continued periods of irradiation. It is suggested that this method of inducing loss of Gram positiveness may prove to be very useful in studying mechanism of Gram stain.—*PHEA*

Testing of Water for the Presence of the Intestinal Bacillus. M. G. Kichenko, L. E. Korsh & N. G. Kichenko. Gigiena i Sanit., **1952**:12:12. New culture medium is suggested for estn. of *Esch. coli*. This is meat-peptone agar with pH 7.4–7.6, which is mixed with 1% lactose and 0.1% glucose and 5% bile. To 1 l of agar are added 5% alc. rosolic acid (2 ml) and 2 ml 1–2% alc. bromothymol blue. Brown-red color of medium changes to yellow in acids and red in alkalies. Typical colon bacilli yield acids from above sugars and yellow color is readily observed.—*CA*

OTHER ARTICLES NOTED

Recent articles of interest, not abstracted, are listed below.

Protecting Underground Utilities Located in Arctic Regions. C. H. Billings. Wtr. & Sew. Wks., **100**:441 (Nov. '53).

How to Buy and Use Lime as a Neutralizing Agent. R. D. Hoak. Wtr. & Sew. Wks., **100**:468 (Dec. '53).

Charting a Course for the Future of Stream Pollution Control. K. S. Watson. Wtr. & Sew. Wks., **100**:421 (Nov. '53).

Recovery of Polluted Streams. C. J. Velz. Wtr. & Sew. Wks., **100**:495 (Dec. '53).

WITH

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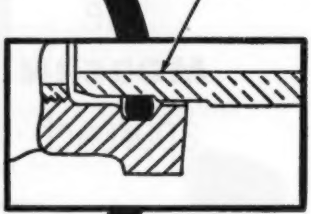

Smith A.W.W.A. Specifications Gate Valves (4" through 12") and Smith Fire Hydrants (with 4" and 6" connecting joints) are now available with Ring-Tite joints for direct connection to Johns-Manville Class 150 Transite Ring-Tite Pipe. No special fittings or extra joint material needed. Installation is quick, easy, economical, and joints can be made in any weather and in roughest terrain. Tight, flexible joints are assured. At low pressures, radial compression of rubber rings seals the joint; increasing pressure wedges the rings—the higher the pressure the tighter the seal. For complete details write for Bulletin RT 53.

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THE A.P. SMITH MFG. CO.

EAST ORANGE, NEW JERSEY

The Reading Meter

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American Standard Cast-Iron Screwed Drainage Fittings—ASA B16.12–1953. *American Society of Mechanical Engineers, 29 W. 39th St., New York 18, N.Y. (1953) 16 pp.; paperbound; $1*

This standard, approved by ASA on Sept. 11, 1953, supersedes the 1942 document.

Sanitation Guide for Small Water Systems. *California Dept. of Public Health, 760 Market St., San Francisco 2, Calif. (1953) 3 pp.; paperbound; free*

These standards were prepared at the request of the California Conference of Local Health Officers, as a guide to the issuance of water supply permits. They present general safety principles for water sources, storage, and distribution, and also discuss water quality.

Regulations Relating to Cross-Connections. *California Dept. of Public Health, 760 Market St., San Francisco 2, Calif. (1953) 6 pp.; paperbound; free*

This reprint contains Sections 7583 through 7622 of the California Administrative Code, Title 17, Public Health, dealing with cross-connection control.

Regulations Relating to Bottled Water. *California Dept. of Public Health, 760 Market St., San Francisco 2, Calif. (1953) 5 pp.; paperbound; free*

A reprint of Section 7695 through 7701 of the California Administrative Code, Title 17, Public Health, dealing with bottled water.

Service Lines

Mueller Water Works Catalog W-96, the company's new and complete catalog which supersedes all previous editions, has just been issued. The book is divided into 19 sections for ease in reference, and is fully illustrated with photographs and parts drawings. Valves, stops, hydrants, regulators, tapping and repair sleeves, valve boxes, and service pipe and fittings are among the products described. Copies may be obtained from Dept. A-36, Mueller Co., Decatur, Ill.

A slide rule and calculator for determining the quantity of coal-tar enamel (in pounds per 1,000 ft or gallons per mile) required for protecting pipe is being distributed by Reilly Tar & Chemical Corp., 1615 Merchants Bank Bldg., Indianapolis 4, Ind.

Chemicals used in water, sewage and waste treatment are indexed in a 16-page booklet, Technical Bulletin 10-K12, distributed by Omega Machine Co., 345 Harris Ave., Providence, R.I. Characteristics, commercial forms available, and various types of application data are provided for 50 chemicals used for water or waste treatment. Specific information on feeding is provided.

Plastics Equipment Reference Sheets, a series of data sheets reprinted from Chemical Engineering Progress, are being distributed by Atlas Mineral Products Co., Mertztown, Pa., the firm with which the authors—Raymond B. Seymour and Earl A. Erich—are connected. The sheets detail the characteristics and commercial form of a number of plastics, and grade the corrosion resistance of each to a variety of chemicals and environments.

"Handbook on Demineralizing," a 40-page information bulletin—No. 5800—on the principles and applications of ion exchange for water treatment, is being distributed by Cochrane Corp., 17th St. below Allegheny Ave., Philadelphia 32, Pa. Details are provided on costs, methods, and results obtainable.

Treatment chemical characteristics are tabulated in a chart, TA-1021-C, giving the trade names, formulas, solubility, strength, uses, and other information on 19 chemicals suitable for application in dry chemical feeders for water, waste, or sewage treatment. Copies are available from Wallace & Tiernan, Box 178, Newark 1, N.J.

Cooling water treatment control is the subject of a 4-page folder, Instrumentation Data Sheet 9.6–7, offered by the Industrial Div., Minneapolis-Honeywell Regulator Co., Wayne & Windrim Aves., Philadelphia 44, Pa.

Spraying systems and industrial spray nozzles are the subject of catalog 24 of Spraying Systems Co., 3201 Randolph St., Bellwood, Ill. The 48 pages of descriptive information include sizes, flow rates, design and materials.

Cold galvanizing with a compound known as Galvicon is said to set up true electrical conductivity between coating and base metal, thus affording cathodic protection. A descriptive folder may be obtained from Galvicon Corp., 40 W. 29th St., New York 1, N.Y.

A chlorinated rubber protective coating known as Parlon is described in a 20-page bulletin available from Hercules Powder Co., Wilmington 99, Del. The material is available as a concrete and metal finish in formulations designed to give protection against various types of wear or attack.

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A low rate disk feeder is announced by Omega Machine Co., 345 Harris Ave., Providence 1, R.I. The model 50 feeder requires a floor space of only 22 by 27 in., offers a feed range from 20 to 1,700 cu in. per hour. A four-page folder gives details.

Gasket, pipe joint, and other sealing compounds are listed in the 1954 catalog of Permatex Industrial Div., 1702 Ave. Y, Brooklyn 35, N.Y. Eight new chemical sealants for special applications are included.

Steel tanks and vessels for water storage are described and illustrated in a 20-page booklet, Catalog H_2O, offered by Hammond Iron Works, 630—5th Ave., New York 20, N.Y. Tables of sizes and capacities are included, as well as illustrations of Aldrich filtration and purification units erected by the company.

Reprints of a paper on the design and operation of controlled volume chemical pumps are offered by the Milton Roy Co., Station H, 1300 E. Mermaid Lane, Philadelphia 18, Pa., as Technical Paper No. 57.

A catalog of process control instruments made by Fischer & Porter Co., 924 Jacksonville Rd., Hatboro, Pa., lists a selection of flow meters, recorders, controllers, pressure regulators, sight flow indicators, and chemical feeders. The 20-page booklet is offered as Catalog 2.

The Principles of "Hydraulic Power" are explained in a folder issued by Henry Mfg. Co., 1700 N. Clay St., Topeka, Kan. The explanation goes into the elements of hydraulics and offers a simplified version of the way the principles are applied in earth moving machinery.

Cooling tower reduction drives utilizing worm gears, spiral bevel, or helical spiral bevel units are the subject of Catalog CT-53 distributed by Philadelphia Gear Works, Erie Ave. & G St., Philadelphia 34, Pa.

Section Meetings

North Carolina Section: The thirty-third annual joint meeting of the North Carolina Section with the North Carolina Sewage and Industrial Waste Assn. was held at the Sheraton Hotel in High Point, N.C., on Nov. 9–11, 1953. The total registration for the meeting was 294, which represents the largest registration at any previous meeting held by the Associations, except at the High Point Convention in 1929 when 303 registered.

At the general session Monday morning, Mayor George Covington presented the "keys to the city" to M. B. Cunningham, AWWA president; L. J. Fontenelli, FSIWA president; and Joe L. Greenlee, chairman of the Section and N.C. Sewage and Industrial Waste Assn. The remainder of the morning session was devoted to a paper entitled "Historical Development of the High Point Water Supply," presented by William F. Freeman of High Point.

The first part of the afternoon program was devoted to a water supply symposium, presided over by Charles Smallwood Jr. of North Carolina State College, who began with "Calculation of Storage Requirements for Water Supply," written by himself, H. Wyndham, of Pierson & Whitman, and D. N. Cote, a graduate student. "Water Inventories as Related to Development" was the title of a paper by E. B. Rice, district engineer for the Geological Survey at Raleigh; and "Use of Polluted Water Sources for Emergency Water Supplies" was discussed by J. M. Jarrett, director of the San. Eng. Div., State Board of Health, Raleigh.

The afternoon session was concluded by an address on the subject "American Water Works Association Program for 1953," which was presented by President Cunningham.

The Honorary Luncheon on Tuesday followed a sewage works session and was in turn followed by addresses by M. B. Cunningham and Louis J. Fontenelli. The afternoon was devoted to recreation and scheduled visits to a local hosiery mill and furniture plant.

Tuesday evening featured the Annual Banquet and Dance. Chairman Greenlee presided at these functions, and President Cunningham presented AWWA Life Membership certificates to H. G. Baity, T. C. Heyward Sr., and immediate Past-Chairman H. F. Davis. The nominee for the George Warren Fuller Award was revealed to be Wade G. Brown, assistant superintendent of the Durham Water and Sewer Dept. Harry J. Siebert of Charlotte and Stanford E. Harris of Winston-Salem received the Maffitt Membership Cup and the Ludlow Trophy for having obtained the greatest number of new members for the AWWA Section and the NCSIWA, respectively. The principal address was delivered by R. L. Patton, superintendent of Burke County Schools, Morganton.

Wade G. Brown presided over the Wednesday morning session, during which progress reports were given on activities of current interest. E. C. Hubbard, executive secretary of the State Stream Sanitation Committee, Raleigh, reported on "Progress of Stream Classification in North Carolina"; Emil T. Chanlett, of the School of Public Health, Univ. of North Carolina, Chapel Hill, gave a "Progress Report on Fluoridation Practices in North Carolina"; and Nelson L. Nemerow, professor of sanitary engineer-

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Section Meetings

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ing, North Carolina State College, Raleigh, gave a "Progress Report on Industrial Waste Research in North Carolina." A very informative paper was presented by L. S. Dukes, Water Dept., Charlotte, on the bichromate reflux method.

The meeting was concluded by the annual business meeting, at which time officers for the coming year were elected and committee reports were presented. New officers elected were: J. M. Jarrett, Raleigh, chairman; Stanford E. Harris, Winston-Salem, vice-chairman; E. C. Hubbard, Raleigh, secretary-treasurer; Harry J. Siebert, Charlotte, trustee; and J. Wilson Setzer, Gastonia, chairman of nominating committee.

E. C. HUBBARD
Secretary-Treasurer

Virginia Section: The twentieth annual conference of the Virginia Section was held in Roanoke on November 4–6, 1953, with headquarters at the Hotel Roanoke. A total of 226 members and guests were registered.

Technical sessions were started by Chairman L. R. McClung, who welcomed the group and urged participation in discussion of subjects that were to follow.

Robert G. Martin, assistant chief, Div. of Fish, Commission of Game and Inland Fisheries, Richmond, discussed "Algae Control and Fish Life in Fresh Water Reservoirs." "Virginia's Annual Short School for Water Works Personnel" was the subject of the next paper by Linvil G. Rich, associate professor of sanitary engineering, Virginia Polytechnic Inst., Blacksburg, Va. The schools have now been put on a permanent basis in Virginia and are to be held at Virginia Polytechnic Inst., Blacksburg.

Charles E. Moore, engineer in charge of construction, Water Dept., Roanoke, delivered a paper on "Automatic Booster Stations." In his talk he described two recent installations made in Roanoke. This paper was discussed by L. A. Geupel, engineer, R. Stuart Royer & Assoc., Richmond, Va. Considerable discussion

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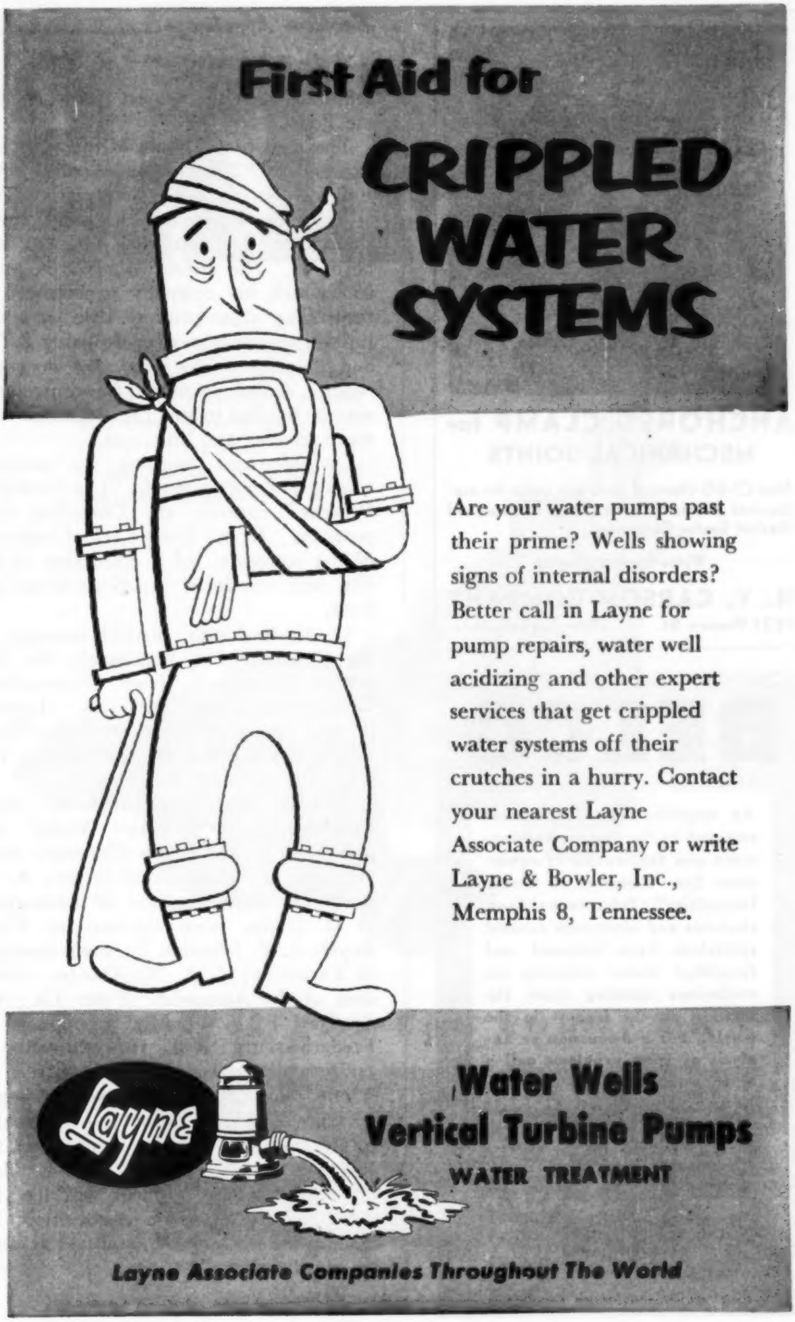
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Section Meetings

(*Continued from page 86 P&R*)

followed when the subject was opened to the floor.

The final presentation of the afternoon was entitled "The Practical Application of E.T.S.-Chlorine to Air Conditioning Water" by B. G. Switzer, assistant power supervisor, E. I. du Pont Co., Waynesboro, Va. The author described the problem which his company experienced in controlling algae prior to 1948, at which time experiments were made, using E.T.S. and chlorine. The treatment does not require skilled personnel for control and may be applied to small installations with much success and little cost.

On Thursday morning the program was opened with a movie, "The Centriline Process," courtesy the Centriline Corporation. K. K. Kirwin, chief engineer of the company, led a discussion of the film and answered questions from the floor.

H. E. Beckwith, district manager of the Pitometer Co., Pittsburgh, Pa., discussed "Newport News' Fifteen-Year Distribution Plan." W. B. Harman, general manager, Newport News Water Works Commission, Newport News, Va., led a discussion of the paper.

A most interesting panel on "Some Problems of Small Water Works" was presented by W. Frank Chapman, town manager of Salem, panel leader; A. K. Roop Jr., town manager of Lexington; J. D. Rives, town engineer of Front Royal; L. Z. Johnston Jr., town manager of Farmville; J. A. McWhorter, President of the Annandale Water Co., Annandale; F. F. Funk, city engineer of Fredericksburg; R. L. Howe, director of sanitary districts, Henrico County.

The Thursday afternoon session began with the showing of a movie—"Corrosion in Action"—through the courtesy of the International Nickel Co. Dale L. Maffitt, AWWA vice-president, described the many activities of the Association, including the numerous committees at work on specifications.

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Section Meetings

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The next topic of the afternoon session was a panel discussion on "Cross-Connections." A report on this subject was prepared by M. C. Smith, chief water and research engineer of the Dept. of Public Utilities, Richmond, and E. C. Meredith, assistant director, Div. of Engineering, State Dept. of Health, Richmond. Kenneth J. Carl, engineer, National Board of Fire Underwriters, New York, and Linn Enslow, vice-president and editor, *Water and Sewage Works*, New York, discussed the report and also criticized certain parts of it. No formal action was taken on adoption of the report at this time. The final presentation of the afternoon was presented jointly by Harry N. Lowe, Sanitary Engineering Branch, and Richard P. Schmitt, chief, Water Purification Section of the Engineer Research and Development Labs., Fort Belvoir, Va. Their subject was entitled "Experience With Solids-Contact Clarifiers in Southeastern Virginia."

The final session on Friday morning, November 6, was begun with showing of a movie, "The Manufacture of Cast-Iron Pressure Pipe," by courtesy of the Cast Iron Pipe Research Assn. The presentation and introduction of the film was by W. N. Hammerstrom, eastern sales manager, Glamorgan Pipe & Foundry Co., Lynchburg. A paper entitled "Water Supplies in Emergencies" was presented by H. P. Kramer, associate chief, Specialized Sanitation Training Section, Environmental Health Center, Cincinnati. The meeting was terminated with a symposium on "New Water Works in Virginia" led by Richard Messer, director, Div. of Engineering, State Dept. of Health, Richmond. W. H. Swindell described the new water filtration plant recently completed for Clarksville; W. F. Middleton discussed the new 2-mgd filter plant now under construction at Emporia; Robert B. McNutt described the new water supply for South Hill; and W. C. Perrow described Chesterfield County's new 1-mgd filtration plant.

The annual banquet Thursday evening, always a highlight of the meeting, was well attended. Following the banquet, H. E. Lordley, chairman of the Fuller Award Committee, announced to the group that the committee had by unanimous decision chosen Howard A. Johnson, superintendent of filtration, Danville, as its nominee.

Social events, aside from the annual banquet, included dinner entertainment followed by a dance. A luncheon for the ladies was held at Yearly Haven, Bennett Springs. These activities were graciously handled by the Local Arrangements Committee and the Club Room Committee, representing the manufacturers and their representatives present.

J. P. Kavanagh
Secretary-Treasurer

California Section: The 34th annual conference of the California Section was held at the Palace Hotel in San Francisco October 27–30 and was presided over by Chairman Laurance E. Goit. The convention began with a men's golf tournament at the San Francisco Golf Club arranged by Claude T. Faw.

The first general session opened on Wednesday afternoon with a welcoming address by Mayor Elmer E. Robinson of San Francisco. The first paper, titled "Liquid Assets of the East Bay Municipal Utility District," was presented by John W. McFarland, manager of the District. He was followed by E. A. Reinke, chief, Bureau of Sanitary Engineering, State Dept. of Public Health, who talked on "Protection of the Sanitary Quality of Water in the Distribution Mains." M. J. Shelton, general manager and chief engineer, La Mesa, Lemon Grove & Spring Valley Irrigation Dist., presented a paper on "Improvements Through Better Administrative Practices."

The "All-Divisions Banquet" was held on Wednesday night. The Fuller Award committee announced its selection of Walter O. Weight and the California

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Section Elliott Award was presented to H. Christopher Medbery. President M. B. Cunningham awarded Life Membership certificates to J. D. DeCosta, A. Kempkey, W. F. Langelier, and E. A. Reinke. A short address by President Cunningham on Association activities concluded the evening's program.

On Thursday separate sessions were held by the Water Supply & Distribution, Water Purification, and Business Management Divisions under the chairmanship of Carl A. Lauenstein, Lee Streicher, and John W. McFarland, respectively. The Water Supply & Distribution Div. program included the following papers: "Construction Joints in Concrete Floors of Reservoirs" by L. A. Hosegood, San Bernardino; "New Developments in Water Well Photography" by Claude Laval Jr., Fresno; "Water Works Problems in Small Towns" by John C. Hayes, Arcadia; "Cutting Pipe in Place" by O. G. Goldman, San Francisco; "Design and Use of Precast Reservoir Roofs" by Carlos Yerby, East Bay Municipal Utility Dist., with a discussion by Paul Beermann, San Diego; "Pumping With Internal Combustion Engines" by C. A. Garnier, Puente, with a discussion by K. H. Beard, Modesto.

The Water Purification Division session comprised the following papers: "Tenth Edition of *Standard Methods*" by Louise Carty, Los Angeles; "Use of Ion Exchange Resins in Water Analysis" by Remo Navone, State Dept. of Public Health; "Recent Applications of the Amperometric Titrator and the Chlorine Residual Recorder" by Morley Weir, Wallace & Tiernan Sales Corp.; "Carrying Free Residual Chlorination Through a Water Distribution System" by George Y. Blair, Palo Alto; "Problems Faced by Irrigation Districts Serving Domestic Water" by Loy Flor, La Mesa, Lemon Grove and Spring Valley Irrigation Dist.; "Control of Sand in Water Supply Systems" by John R. Rossum, California Water Service Company (see this issue, p. 123), and "Watershed Control for Domestic Water Supply Reservoirs," a panel discussion by R. E. Dodson, San Diego, and Gordon Laverty, East Bay Municipal Utility Dist.

The Business Management Division session featured a number of speakers who were from organizations outside the water works field but who, as specialists in their respective businesses, added new points of view to water works matters. The session included a talk on "Looking Ahead in Labor Relations" by J. Hunter Clark of J. Hunter Clark and Staff, Industrial Engineers, Oakland, a paper on "Public Relations and Opportunity for Your Water Department" by A. O. Putnam, Director of Public Relations, Layne & Bowler, and a talk on "Management Attitudes Toward Safety Engineering" by Kenneth N. Beadle, Director of Safety, Pacific Intermountain Express Co. and 1952 winner of the Marcus Dow Award for the outstanding safety director in the United States; a paper titled "Does Your Accountant Prepare Costs for Management?" by D. R. Malcolm Jr., Price, Waterhouse and Co., and a panel discussion on "How to Lead Men" participated in by William J. Stephens, East Bay Municipal Utility Dist.; Ben Haggott, Palos Verdes Water Co.; Lloyd D. Luckmann, San Francisco; Otto E. Never, Crescent City; and George C. Sopp, Los Angeles.

The general session on Friday included a paper on "Underground Movement of Chemical and Bacterial Pollutants" by P. H. McGauhey, R. S. Butler, and G. T. Orlob of the University of California (see this issue, p. 97) and a paper on "Water Spreading Operations in the San Gabriel Valley" by Finley Laverty (see this issue, p. 112). Thomas T. Quigley, Director of Water & Sewerage Industry and Utilities Div., U.S. Dept. of Commerce, reported on the activities of his office. California State Engineer A. D. Edmonston presented a paper on "California Water Problems." The afternoon session began with the business meeting at which time H. Christopher Medbery was elected

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Section Meetings

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chairman for the coming year with Louis J. Alexander to serve as vice-chairman and Henry F. Jerauld to serve as secretary-treasurer for the next two years. Burton S. Grant was elected to serve for three years, beginning in May 1954, as national director. H. H. Harris, A. F. Poulter, and John C. Luthin were the newly elected members to the Executive Committee.

Following the business meeting, President Morrison B. Cunningham gave a talk on "AWWA in 1953." John S. Longwell reviewed the status of California's civil defense plans. The program concluded with a talk on "The Effect of Federal Legislation on California Water Projects" by Clair Engle, California congressman.

The annual banquet sponsored by the Water & Sewage Works Manufacturers Assn. was held on Friday night. The prizes for the men and women's golf tournaments were awarded and the evening was concluded with entertainment and dancing.

The ladies' program for the convention arranged by Mrs. Joseph D. DeCosta included a tea and Chinese fashion show on Wednesday, a golf tournament, luncheon, and book review at the Orinda Country Club on Thursday, and a guided shopping tour in San Francisco on Friday.

The conference registration was 1,126, somewhat lower than the record attendance of 1,434 of the 1952 Pasadena conference. Fifty manufacturing firms were represented, and the exhibits under the chairmanship of Jack Peterson were exceptionally good and well attended. One of the particularly distinctive features of the meeting was that all sessions, the exhibits, and banquets were held in the Palace Hotel which added to the convenience of those who attended.

JOHN C. LUTHIN
Secretary-Treasurer

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R. D. Wood Co.

Gears, Speed Reducing:
DeLaval Steam Turbine Co.
Philadelphia Gear Works, Inc.

Glass Standards—Colorimetric Analysis Equipment:
Hellige, Inc.
Klett Mfg. Co.
Wallace & Tiernan Co., Inc.

Goosenecks (with or without Corporation Stops):
James B. Clow & Sons
Hays Mfg. Co.
James Jones Co.
Mueller Co.

Hydrants:
James B. Clow & Sons
Darling Valve & Mfg. Co.
M. Greenberg's Sons
James Jones Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pacific States Cast Iron Pipe Co.
A. P. Smith Mfg. Co.
Rensselaer Valve Co.
R. D. Wood Co.

Hydrogen Ion Equipment:
Hellige, Inc.
Wallace & Tiernan Co., Inc.

Ion Exchange Materials:
Cochrane Corp.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Rohm & Haas Co.

Iron Removal Plants:
American Well Works
Belco Industrial Equipment Div.
Chain Belt Co.
Cochrane Corp.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Welsbach Corp., Ozone Processes Div.

Jointing Materials:
Atlas Mineral Products Co.
Hydraulic Development Corp.
Johns-Manville Corp.
Leadite Co., Inc.
Northrop & Co., Inc.

Joints, Mechanical, Pipe:
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Dresser Mfg. Div.
McWane Cast Iron Pipe Co.
Pacific States Cast Iron Pipe Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Leak Detectors:
Jos. G. Pollard Co., Inc.

Lime Slakers and Feeders:
Dorr Co.
Infilco Inc.
Omega Machine Co. (Div., B-I-F Industries)
Permutit Co.

Magnetic Dipping Needles:
W. S. Darley & Co.

Meter Boxes:
Ford Meter Box Co.
Pittsburgh Equitable Meter Div.

Meter Couplings and Yokes:
Badger Meter Mfg. Co.
Dresser Mfg. Div.
Ford Meter Box Co.
Hays Mfg. Co.
Hersey Mfg. Co.
James Jones Co.
Mueller Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Smith-Blair, Inc.
Welsbach Corp., Kitson Valve Div.
Worthington-Gamon Meter Co.

Meter Reading and Record Books:
Badger Meter Mfg. Co.

Meter Testers:
Badger Meter Mfg. Co.
Ford Meter Box Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.

Meters, Domestic:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Builders-Providence, Inc.
Infilco Inc.
Simplex Valve & Meter Co.
Sparling Meter Co., Inc.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Buffalo Meter Co.
Builders-Providence, Inc.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
Simplex Valve & Meter Co.
Sparling Meter Co., Inc.
Well Machinery & Supply Co.
Worthington-Gamon Meter Co.

Mixing Equipment:
Chain Belt Co.
Infilco Inc.
Walker Process Equipment, Inc.

Ozonation Equipment:
Welsbach Corp., Ozone Processes Div.

Pipe, Asbestos-Cement:
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
American Brass Co.

Pipe, Cast Iron (and Fittings):
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Crane Co.
McWane Cast Iron Pipe Co.
Pacific States Cast Iron Pipe Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
Cast Iron Pipe Research Assn.
James B. Clow & Sons
McWane Cast Iron Pipe Co.
Pacific States Cast Iron Pipe Co.
Southern Pipe & Casing Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe Coatings and Linings:
The Barrett Div.
Cast Iron Pipe Research Assn.
Centriline Corp.
Koppers Co., Inc.
Pipelife, Inc.
Reilly Tar & Chemical Corp.
Southern Pipe & Casing Co.

Pipe, Concrete:
American Concrete Pressure Pipe Assn.
American Pipe & Construction Co.
Lock Joint Pipe Co.
Universal Concrete Pipe Co.

Pipe, Copper:
American Brass Co.

Pipe Cutting Machines:
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Pipe Jointing Materials: see Jointing Materials

Pipe Locators:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pipe, Plastic:
Carlon Products Corp.

Pipe, Steel:
American Locomotive Co.
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.
Southern Pipe & Casing Co.

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potentiometers:
Hellige, Inc.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Economy Pumps, Inc.
Morse Bros. Mchy. Co.

Pumps, Chemical Feed:
Infilco Inc.
Proportioneers, Inc.
Wallace & Tiernan Co., Inc.

Pumps, Deep Well:
American Well Works
Layne & Bowler, Inc.

Pumps, Diaphragm:
Dorr Co.
Morse Bros. Mchy. Co.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Economy Pumps, Inc.

Pumps, Sump:
DeLaval Steam Turbine Co.
Economy Pumps, Inc.

Pumps, Turbine:
DeLaval Steam Turbine Co.
Layne & Bowler, Inc.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Permutit Co.
Wallace & Tiernan Co., Inc.

Recording Instruments:
Infilco Inc.
Sparling Meter Co., Inc.
Wallace & Tiernan Co., Inc.

Reservoirs, Steel:
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves; see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
Permutit Co.

Soda Ash:
Solvay Process Div.

Sodium Hexametaphosphate:
Blockson Chemical Co.
Calgon, Inc.

Softeners:
Belco Industrial Equipment Div.
Cochrane Corp.
Dorr Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon, Inc.
Cochrane Corp.
Infilco Inc.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Steel Plate Construction:
American Locomotive Co.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Hays Mfg. Co.
James Jones Co.
Mueller Co.
Welsbach Corp., Kitson Valve Div.

Storage Tanks; see Tanks

Strainers, Suction:
James B. Clow & Sons
M. Greenberg's Sons
Johnson, Edward E., Inc.
R. D. Wood Co.

Surface Wash Equipment:
Cochrane Corp.
Permutit Co.

Swimming Pool Sterilization:
Everson Mfg. Corp.
Omega Machine Co. (Div., B-I-F Industries)
Proportioneers, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

Tanks, Steel:
American Locomotive Co.
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
A. P. Smith Mfg. Co.
Hays Mfg. Co.
Mueller Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.
Welsbach Corp., Kitson Valve Div.

Taste and Odor Removal:
Cochrane Corp.
Industrial Chemical Sales Div.
Infilco Inc.
Permutit Co.
Proportioneers, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Hellige, Inc.
Wallace & Tiernan Co., Inc.

Turbines, Steam:
DeLaval Steam Turbine Co.

Turbines, Water:
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Pacific States Cast Iron Pipe Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valve-Inserting Machines:
A. P. Smith Mfg. Co.

Valves, Altitude:
Davis Mfg. Co.
Ross Valve Mfg. Co., Inc.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Backflow Engineering & Equipment Co.
James B. Clow & Sons
Crane Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey Mfg. Co.

Valves, Electrically Operated:
Belco Industrial Equipment Div.
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Davis Mfg. Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Valves, Float:
James B. Clow & Sons
Davis Mfg. Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
James Jones Co.
Kennedy Valve Mfg. Co
M & H Valve & Fittings Co.
Mueller Co.
Pacific States Cast Iron Pipe Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Davis Mfg. Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Philadelphia Gear Works, Inc.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Large Diameter:
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
Davis Mfg. Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:
Crane Co.
Davis Mfg. Co.
Mueller Co.
Ross Valve Mfg. Co.

Valves, Swing Check:
James B. Clow & Sons
Crane Co.
Darling Valve & Mfg. Co.
M. Greenberg's Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Waterproofing
Inertol Co., Inc.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
Hellige, Inc.
Wallace & Tiernan Co., Inc.

Water Treatment Plants:
Allis-Chalmers Mfg. Co.
American Well Works
Belco Industrial Equipment Div.
Chain Belt Co.
Chicago Bridge & Iron Co.
Cochrane Corp.
Dorr Co.
Graver Water Conditioning Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Co., Inc.
Welsbach Corp., Ozone Processes Div.

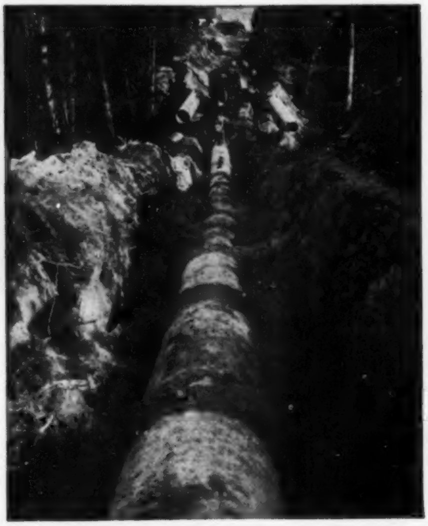
Well Drilling Contractors:
Layne & Bowler, Inc.

Well Screens
Johnson, Edward E., Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite; see Ion Exchange Materials

Norfolk Prefers Concrete Pressure Pipe



Since 1921, Norfolk, Virginia, has been specifying concrete pressure pipe for its water supply and distribution system. Over 450,000 feet of pipe is now in use. Diameters range from 20" to 48".

Still in excellent condition is the 31,700 feet of concrete pressure pipe laid in 1921. There has been no necessity to take this pipeline out of service for any maintenance work; nor has the pipeline suffered from any trouble due to electrolytic action. This pipe is now carrying water at the same high capacity as when it was installed.

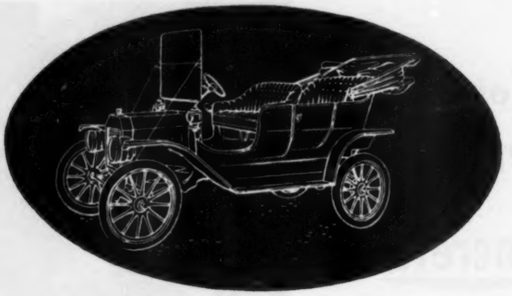
If your city wants pipe with an assured high-carrying capacity, decade after decade...if long term economy is a necessity . . . then look into the advantages of concrete pressure pipe when you plan your next transmission or distribution lines.

Member companies manufacture concrete pressure pipe in accordance with nationally recognized specifications

Concrete PRESSURE Pipe

AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION
228 North LaSalle Street
Chicago 1, Illinois

WATER FOR GENERATIONS TO COME

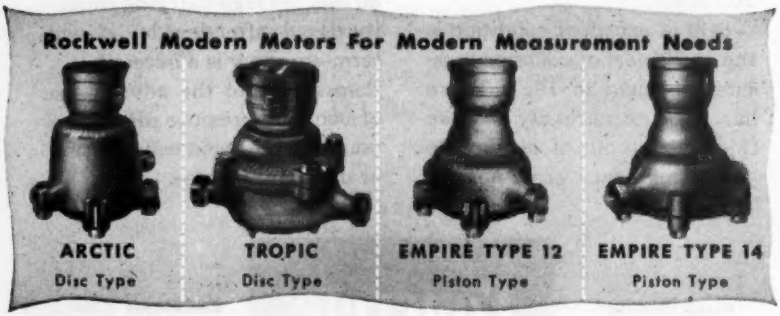


Would you keep on repairing a 1912 car?

There are many thousands of meters purchased thirty or more years ago still in service. They are being periodically repaired—frequently rebuilt from the old meter case up. This practice can impose a tremendous burden of trouble and expense on a water utility, especially at today's higher labor rates.

Many management men, with an eye towards costs, know these facts. They know that a new Rockwell meter will measure 95% of the ¼ gpm or smaller flows as against 90% or less for a repaired meter. They realize that if the cost of repairing an old meter amounts to half the cost of a new meter, they are better off to buy a new one.

Here at Rockwell we've worked constantly for improvement and our meters show it. They will measure at top accuracy for longer periods at lower costs. You can buy them with full confidence in their ability to cut your operating and maintenance costs and increase your revenue.



The Symbol for Service, Quality and Performance in Water Meters

ROCKWELL MANUFACTURING COMPANY

PITTSBURGH 8, PA. Atlanta Boston Chicago Houston Los Angeles N. Kansas City New York Philadelphia Pittsburgh San Francisco Seattle Tulsa

In Canada: Peacock Brothers Limited

LEADITE

Trade Mark Registered U. S. Pat. Office

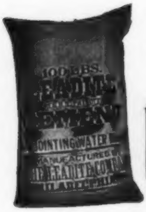
Jointed for . . . Permanence with LEADITE

Generally speaking, most Water Mains are buried beneath the Earth's surface, to be forgotten,—they are to a large extent, laid for permanency. Not only must the pipe itself be dependable and long lived,—but the joints also must be tight, flexible, and long lived,—else leaky joints are apt to cause the great expense of digging up well-paved streets, beautiful parks and estates, etc.

Thus the "jointing material" used for bell and spigot Water Mains MUST BE GOOD,—MUST BE DEPENDABLE,—and that is just why so many Engineers, Water Works Men and Contractors aim to PLAY ABSOLUTELY SAFE, by specifying and using LEADITE.

Time has proven that LEADITE not only makes a tight durable joint,—but that it improves with age.

The pioneer self-caulking material for c. i. pipe.
Tested and used for over 40 years.
Saves at least 75%



THE LEADITE COMPANY
Girard Trust Co. Bldg. Philadelphia, Pa.

No Caulking